QUINNIPIAC RIVER BASIN

BETHANY, CONNECTICUT

LAKE CHAMBERLAIN DAM
CT 00306

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

WALTHAM, MASS. 02154

AUGUST 1978

UNCLASSIFIED

URITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
REPORT NUMBER CT 00306	2. GOVT ACCESSION NO. AD A144290	3. RECIPIENT'S CATALOG NUMBER
TITLE (and Subtitle) Lake Chamberlain Dam ATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL AMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT
		6. PERFORMING ORG. REPORT NUMBER
AUTHOR(s) .S. ARMY CORPS OF ENGINEERS EW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
CONTROLLING OFFICE NAME AND ADDRESS EPT. OF THE ARMY, CORPS OF ENGINEERS EW ENGLAND DIVISION, NEDED 24 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE August 1978
		13. NUMBER OF PAGES 70
MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) UNCLASSIFIED
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

DISTRIBUTION STATEMENT (of this Report)

PPROVAL FOR PUBLIC RELEASE: DISTRIBUTION UNLIMITED

DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

SUPPLEMENTARY NOTES

over program reads: Phase I Inspection Report, National Dam Inspection Program; owever, the official title of the program is: National Program for Inspection of on-Federal Dams; use cover date for date of report.

KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Quinnipiac River Basin
ethany, Connecticut

ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam is an earthen embankment, with a 53 ft. high masonry rubble corewall, approx- mately 1300± ft. in length and having a maximum height of 88± ft. above the elevation f the original streambed. Based upon visual inspections at the site and past per- ormance, the dam is judged to be in good condition. Based upon the size (intermediate) and hazard classification (high) of the dam in accordance with the orps of Engineers guidelines, the Test Flood will be equivalent to the Probable laximum Flood.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF

NEDED

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

NOV 30 1978

Dear Governor Grasso:

I am forwarding to you a copy of the Lake Chamberlain Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, the New Haven Water Company, Sargent Drive, New Haven, Connecticut 06506, ATTN: Mr. Jack Reynolds, Superintendent Source of Supply.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely yours,

JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

LAKE CHAMBERLAIN DAM

CT 00306

QUINNIPIAC RIVER BASIN
BETHANY, CONNECTICUT

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Inventory Number:	CT 00306
Name of Dam:	LAKE CHAMBERLAIN
State Located:	CONNECTICUT
County Located:	NEW HAVEN
Town Located:	BETHANY
Stream:	SARGENT RIVER
Owner:	NEW HAVEN WATER COMPANY
Date of Inspection:	JUNE 1, 1978
Inspection Team:	MIKE HORTON
	HECTOR MORENO
	GONZALO CASTRO
	DEAN THOMASSON

The dam is an earthen embankment, with a 53 foot high masonry rubble corewall, approximately 1300± feet in length and having a maximum height of 88± feet above the elevation of the original streambed. The maximum width at the top is 22 feet with the downstream slope having a maximum inclination of 2 horizontal to 1 vertical, and the upstream slope a 3 horizontal to 1 vertical maximum inclination, as indicated on the "As-Built" plans. The low level inlet is a 42 inch reinforced concrete pipe which feeds two 30 inch cast iron outlet pipes. The spillway is a 50 foot concrete ogee section located at the left end of the dam. Glen Lake Dam and populated areas of Woodbridge are located one and two miles downstream of the dam, respectively.

Based upon visual inspections at the site and past performance, the dam is judged to be in good condition. No evidence of structural instability was observed, and the condition of the earthen embankment is good. However, there are some areas requiring monitoring and minor maintenance.

Our hydraulics computations, indicate the spillway capacity is 8,100 cubic feet per second, which is in excess of 100 percent of the Test Flood. Based upon the size (Intermediate) and hazard classification (High) of the dam in accordance with Corps of Engineers guidelines, the Test Flood will be equivalent to the Probable Maximum Flood (PMF). Peak inflow to the reservoir is 7,600 cubic feet per second; peak outflow (Test Flood) is 5,500 cubic feet per second. The dam freeboard during the Test Flood is approximately 2.7 feet. The peak failure outflow for the dam if breached would be 251,000 cubic feet per second. A breach of the dam would cause Glen Lake Dam downstream to be overtopped by 20 feet and most likely to fail, causing severe loss of life and damage to property further downstream.

It is recommended that a monthly program for monitoring the seeps which were observed at the downstream face and toe of the dam, be implemented. Locations of exit points of the seeps surfacing downstream of the dam should be ascertained and the potential for piping or boils, as well as required seepage control measures, if any, should be determined.

Shifting of the channel wall at the construction joint on the land side of the spillway should be monitored regulary. An operation and maintenance plan should be instituted.

The above recommendations and remedial measures should be implemented within one year of the owner's receipt of this Phase I Inspection Report.

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

William O. Doll, P.E.
Chief Engineer
Cahn Engineers, Inc.

This Phase I Inspection Report on Lake Chamberlain Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Charles G. Tiersch

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

Fred J. Ravens Jr.

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

Saul Cooper

SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionarly in nature. It would be incorrect to assume that the present condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

TABLE OF CONTENTS

Page

APPENDIX

*See Special Note Appendix Section B Availability of Data

OVERVIEW PHOTO

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS. CAHN ENGINEERS, INC. WALLINGFORD, CONN. ARCHITECT —— ENGINEER	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	LAKE CHAMBERLAIN DAM SARGENT RIVER	BETHANY CONNECTICUT	DATE 6/1/78 CE # 27 531 GC PAGE ix

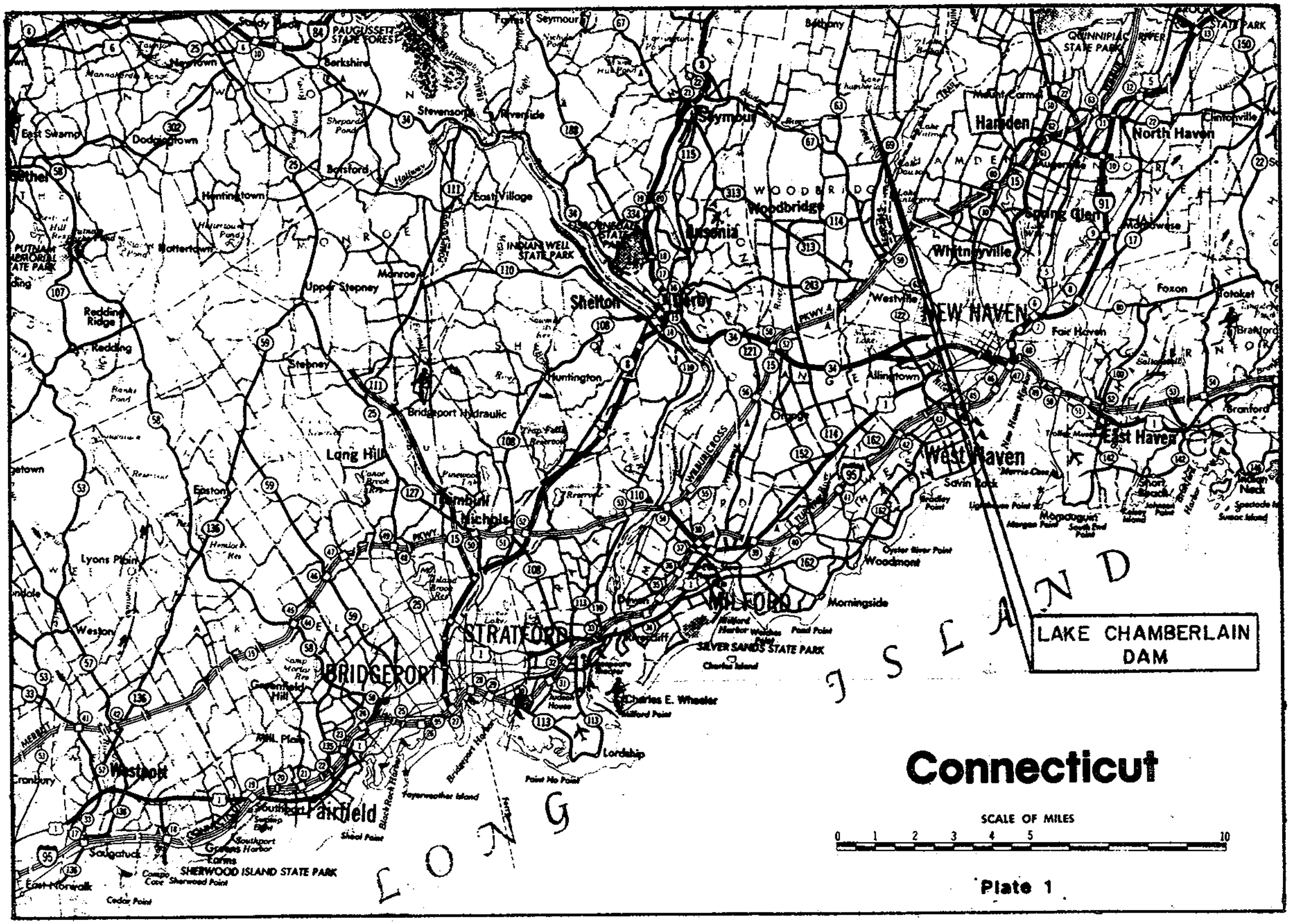
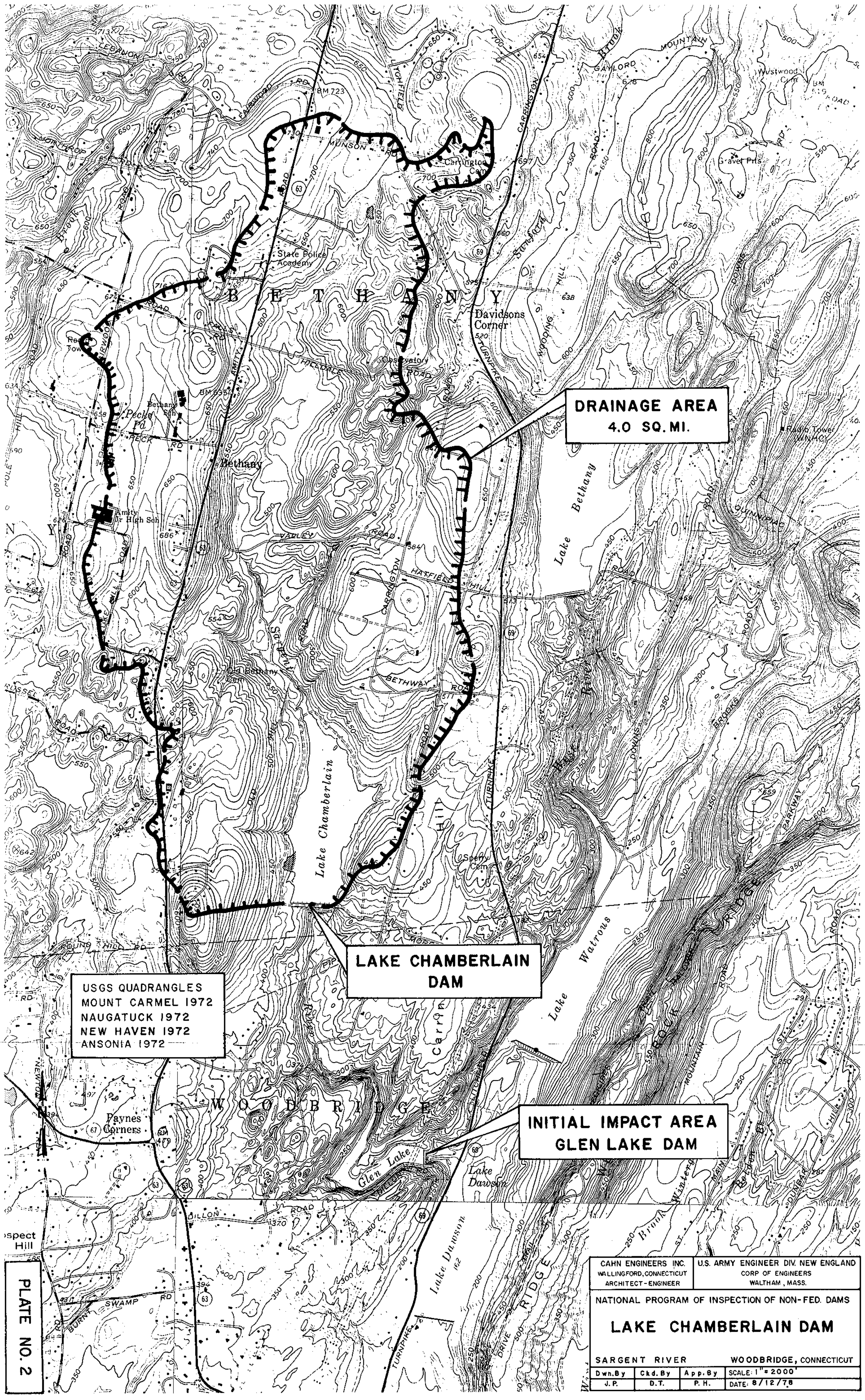

LAKE CHAMBERLAIN DAM
Connecticut
SCALE OF MILES
0 1 2 3 4 5 10
NEW HAVEN
West Haven
East Haven
Hamden
North Haven
Whitneyville
Woodbridge
Ansonia
Seymour
Derby
Shelton
Orange
Allingtown
MILFORD
STRATFORD
BRIDGEPORT
Fairfield
Westport
Trumbull
Nichols
Monroe
Stepney
Upper Stepney
Huntington
Long Hill
Easton
Weston
Lyons Plain
Redding
Redding Ridge
Newtown
Botsford
Berkshire
Stevenson
Riverside
Sandy Hook
PAUGUSSETT STATE FOREST
QUINNIPIAC RIVER STATE PARK
INDIAN WELL STATE PARK
SILVER SANDS STATE PARK
SHERWOOD ISLAND STATE PARK
Saugatuck
East Norwalk
Southport
Greens Farms
Lordship
Woodmont
Morningside
Savin Rock
Fair Haven
Branford
Foxon
Bethany
Mount Carmel
Spring Glen
Montowese
Clintonville
Westville
Dodgingtown
Huntingtown
Bettertown
East Village
Bridgeport Hydraulic
East Swamp
Greenfield Hill
Cranbury
LONG ISLAND

Plate 1


DRAINAGE AREA
4.0 SQ. MI.
LAKE CHAMBERLAIN
DAM
USGS QUADRANGLES
MOUNT CARMEL 1972
NAUGATUCK 1972
NEW HAVEN 1972
ANSONIA 1972
INITIAL IMPACT AREA
GLEN LAKE DAM
BETHANY
WOODBRIDGE
Lake Chamberlain
Lake Bethany
Lake Watrous
Lake Dawson
Glen Lake
State Police Academy
Davidsons Corner
Paynes Corners
CAHN ENGINEERS INC.
WALLINGFORD, CONNECTICUT
ARCHITECT-ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORP OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
LAKE CHAMBERLAIN DAM
SARGENT RIVER
WOODBRIDGE, CONNECTICUT
Dwn. By J. P.
Ckd. By D. T.
App. By P. H.
SCALE: 1" = 2000'
DATE: 8/12/78
PLATE NO. 2

PHASE I INSPECTION REPORT

LAKE CHAMBERLAIN DAM

SECTION I

PROJECT INFORMATION

1.1 General

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the southwestern portion of the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 26, 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0310 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

(1) Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

(2) Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

(3) To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

(1) Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

(2) A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

(3) Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

(4) An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features on the dam which need corrective action and/or further study.

1.2 Description of Project

a. Description of Dam and Appurtenances - The dam is an earthen embankment with a 53 foot high masonry rubble corewall founded on rock, built as part of the original 53 foot high rolled earth dam. It was raised to its present height 88± feet above the original streambed, in 1958. The top has a maximum width of 22 feet and is approximately 1300 ± feet in length. The upstream and downstream slopes are at maximum inclinations of 2 horizontal to 1 vertical and 3 horizontal to 1 vertical, respectively. The spillway is a 50 foot concrete ogee section cut into rock at the left end of the dam. The low level inlet is a 42 inch reinforced concrete pipe which empties into an inlet structure, as do two 30 inch inlets above it. The outlet from the inlet structure consists of two 30 inch cast iron pipes passing through the dam to the downstream outlet structure.

At the spillway crest level, the resevoir area is approximately 115 acres with a useable capacity of approximately 894 million gallons.

The dam is located upstream of the Dawson Lake and Glen Lake Dams, as well as residential developments in the Woodbridge area.

b. Location - The dam is located on the Sargent River in a rural area of the Town of Bethany, about two miles from of the Town of Woodbridge, County of New Haven, State of Connecticut. The dam is shown on the Mt. Carmel USGS Quadrangle Map having coordinates latitude N41° 23' 37" and longitude 72° 59' 19".

c. Size Classification - INTERMEDIATE - The dam has a maximum height of 88± feet from the top to the old streambed, and a storage of 4120 acre feet at the test flood pool, elevation 407.6. According to the Recommended Guidelines, a dam having between 1,000 and 50,000 acre feet of storage is considered to be in the intermediate size range.

d. Hazard Classification - HIGH (Category I) Failure of Chamberlain Dam would cause Glen Lake Dam, approximately 1 mile downstream, to be overtopped by 20 feet and most likely fail, causing severe loss of life and property damage further downstream.

e. Ownership - New Haven Water Company
Sargent Drive
New Haven, Connecticut 06506
Mr. Joseph Jiskra
Mr. Jack Reynolds (203) 624-6671

f. Purpose of Dam - Public water supply reservoir.

g. Design and Construction History - In 1891, the original Lake Chamberlain Dam, was constructed by C.W. Blakeslee and Sons, Inc., as engineered by Henry B. Gorham. The entire dam was founded on rock with a 39 foot spillway cut into rock at the left end of the dam. Two 30-inch cast iron low level inlets were installed through the dam to let water down to Glen Lake as needed. In 1958, the dam was raised 35 feet to its present height by C.W. Blakeslee and Sons, Inc., as engineered by Malcolm Pirnie Engineers. The structure is a compacted earth dam with a side channel spillway, a 50 foot concrete ogee section, cut into rock at the left end of the dam. The upstream slopes of the dam are faced with riprap. The two-30 inch low level lines of the original dam are utilized with new intake and outlet structures.

h. Normal Operational Procedures - Valves are operated as needed during the summer months to supply water to downstream reservoirs when the flow no longer overtops the spillway.

1.3 Pertinent Data

a. Drainage Area - 4.0 square miles. Rolling wooded terrain.

b. Discharge at Dam Site - Maximum water over spillway during August and October 1955 floods - 12" on October 16, 1955. Spillway Capacity at Test Flood Pool Elevation 407.6 - 5500 cfs.

c. <u>Elevation</u> - (Ft. above MSL, U.S.G.S. Datum)

Top of Dam:	410.3
Spillway Crest:	398.3
Streambed @ Center Line of Dam:	322
High Level Intake:	375 and 358
Low Level Intake:	329
Outlet Pipe:	318±

d. <u>Reservoir</u> - Length of Normal Pool: 4500 ft.

Length of Maximum Pool: 4500+ ft.

e. <u>Storage</u> - At Elevation 398.3 — 2740 acre ft.
At Elevation 410.3 (top of dam) — 4120 acre ft.

f. <u>Reservoir Surface</u> -

At Elevation 398.3	115 acres
At Elevation 410.3	115+ acres

g. <u>Dam</u> -

Type:		Compacted/rolled earth with masonry corewall.
Length:	Dam:	1,300 ft.
	Corewall:	710 ft.
Height:		88 feet
Top Width:		22' Minimum - Dam 5' Maximum -Corewall
Side Slope:		2.5 H to 1V upstream 2 H to 1V downstream
Corewall:		Masonry (old dam corewall)
Cutoff:		Foundation on rock - both dam and corewall.

h. <u>Diversion and Regulatory Tunnel</u> - Not Applicable

i. <u>Spillway</u> -

Type:	Concrete ogee weir.
Length of Weir:	50 feet
Crest Elevation:	398.3
Upstream Channel:	17 H to 1V concrete
Downstream Channel:	5.5 H to 1V concrete

j. <u>Regulatory Outlets</u> - 1-42" Low Level Intake
2-30" Feed to Channel

SECTION 2: ENGINEERING DATA

2.1 Design

a. Available Data - The available data provided by the State of Connecticut and the Owner, consists of drawings, correspondence, records, and calculations by the State of Connecticut Water Resources Commission, New Haven Water Company, Philip W. Genovese and Associates, Malcolm Pirnie Engineers, Joseph W. Cone, and others. Considerable data is available with respect to the hydraulic/hydrologic nature and past history of the facility. Pertinent data is included in the Appendix Section B.

b. Design Features - The maps and drawings indicate the design features described previously herein.

c. Design Data - There were no engineering values, assumptions, test results, or calculations available for the original construction or for the 1958 raising. The design data available addresses only the hydraulic/hydrologic characteristcs of the facility.

2.2 Construction

a. Available Data - The available construction drawings consist of a set of plans entitled "As-Built, New Haven Water Company, New Haven, Conn., Chamberlain Dam", by Malcolm Pirnie Engineers, dated July 1958.

b. Construction Considerations - No information was available.

2.3 Operation - No formal operations records exist. Operations were made available for visual inspection by the owner.

2.4 Evaluation

a. Availability - Existing data was provided by the State of Connecticut and the owner. The owner made the operations available for visual inspection.

b. Adequacy - The amount of existing data provided was substantial. However, the amount of detailed engineering data available was inadequate to perform in-depth assessment of the dam. Therefore, the final assessment of this investigation must be based primarily on visual inspection, performance history, hydraulic computations of spillway capacity based upon approximate hydrologic assumptions.

c. Validity - The drawings and correspondence portray the dam substantially as observed during the field inspection.

SECTION 3: VISUAL INSPECTION

3.1 Findings

a. General - The general appearance of the dam is good. However, close inspection reveals some areas requiring minor maintenance.

b. Dam

Upstream Slope - During inspection, the reservoir level was slightly over the spillway; thus only the upper part of the upstream slope was inspected. The upper 4 feet of the slope is grass covered with no evidence of significant erosion. Below the grass-covered zone, there is riprap consisting of stone, ranging in size from about one inch to about 2.5 ft. The riprap appears, in general, in good condition, even though there are some areas where there is some segregation of the smaller and larger stone sizes.

Crest - The crest of the dam contains a gravel roadway with grass-covered berms. No evidence of erosion or cracking was observed along the crest.

Downstream Slope - The downsteam slope is grass covered with no evidence of sloughing or wet spots observed. There has been trespassing of motocycles creating paths over which erosion can eventually develop, even though the erosion at the time of the visual inspection was minor. The darker green area, seen on the upper part of the slope, corresponds to a different type of vegetation cover being tried for higher resistance to trespassers.

A seep was observed along the toe of the slope. Crushed stone was placed in the area of the seep, and water flowing due to the seep covered an area wider than that of the stone. The water appears clear, and no evidence of silt deposition was observed. It appears that the volume of flow increases as the stream travels along the toe, indicating that there is more than one source of water along the toe. The stream eventually discharges into the outlet channel about 20 feet downstream of the outlet structure.

A wet area exists about 100 feet downstream of the toe of the dam. The water flow covers an area about 50 feet wide as it approaches the outlet channel. The wet areas are indicated by the darker green vegetation. Upon close inspection, the water appears to be clear, and no evidence of silt transport was apparent. In the part of the wet area farthest from the outlet channel, a few bedrock exposures were noted. These exposures become more prevalent as one approaches the spillway channel, and the bottom of the spillway channel itself is bedrock.

c. <u>Appurtenant Structures</u> - The outlet structure and gate chamber are in good condition, and the outlet channel is the natural bed of the river. The spillway channel was excavated in bedrock and is in good condition.

3.2 <u>Evaluation</u>

The visual inspection was sufficient to indicate no immediate safety problems. Seeps observed at the toe of the dam and downstream of the dam carry a significant volume of water, although there is no visual evidence of piping. The significance of the seeps has to be analyzed in reference to the zoning of the earth embankment and to the foundation soils and bedrock, as will be discussed in Section 6.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Regulating Procedures

No regulating procedures exist for this dam other than those necessary for providing sufficient water to downstream reservoirs as needed to maintain an adequate public water supply.

4.2 Maintenance of Dam

Water levels in the reservoir are recorded daily and seeps at the toe of the dam are observed periodically and monitored. Any needed maintenance is observed and reported during these visits. During the growing season, the grass is cut regulary. Riprap has been dumped in areas of the seeps, and recent field investigations by the owner or representatives of the owner may result in remedial measures consisting of a system of drains being installed.

4.3 Maintenance of Operating Facilities

The maintenance of the operating facilities is on an as-needed basis. The valves are generally operated during dry seasons to provide water to downstream reservoirs. The valves are usually greased every one to two years.

4.4 Description of Any Warning System in Effect

No formal warning system is in effect. The dam operator reports emergency situations directly to his supervisor.

4.5 Evaluation

Maintenance procedures, as they exist presently, are generally good and should be continued on a regular basis. However, operation and maintenance procedures should be documented on a formal basis to provide accurate records for future reference. A formal warning system should be developed to warn the downstream population of possible emergencies.

5.1 Evaluation of Features

a. Design Data - No computations could be found for the original dam construction. The report on dams owned by the New Haven Water Company by Joseph W. Cone, and the report by Malcolm Pirnie Engineers on effects of the maximum possible storm on the spillways of dams in the West River System, both contain information on the hydraulic/hydrologic computations conducted for the respective reports, which are included in the Appendix Section B.

b. Experience Data - Water generally flows over the spillway from late fall to early summer.

c. Visual Observations - On the dates of our inspections, the spillway was clear and unobstructed. The spillway is spanned by a bridge; however, due to the fact that during the test flood, the dam will still have approximately 2.7 feet of freeboard, the possibility of blockage due to the bridge collecting debris is minimal. It is possible that blockage due to large debris (trees) could occur at the concrete spillway entrance.

Any overtopping will occur first at the dike to the left of the spillway as the elevation at the top of the dike is 2 feet below that of the top of the embankment.

d. Overtopping Potential - The recommended spillway test flood for this high hazard intermediate size dam is the Probable Maximum Flood (PMF). Based upon hydraulics computations, the spillway capacity is 8100 cfs (Appendix D-10). Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges" March 1978, peak inflow to the reservoir is 7600 cfs (Appendix D-9); peak outflow (Test Flood) is 5500 cfs with approximately 2.7 feet of freeboard maintained (Appendix D-12).

e. Spillway Adequacy - The spillway will pass in excess of 100% of the 5500 cfs test flood without overtopping.

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations - From a structural standpoint, the dam, the spillway sidewalls and spillway channel all appear to be stable with no problems indicated. Some shifting of the channel wall has occurred at the construction joint on the landside of the spillway. The intake chamber is also in good condition, and does not appear to have any stability problems.

Visual inspection from a geotechnical standpoint did not disclose any apparent stability problems.

b. Design and Construction Data - The design drawings and specifications of July 1958 for raising the dam indicate some intended zoning for the earth embankment consisting of:

1. "Sandy Material" to an unspecified depth under the upstream and downstream slopes.
2. "Compacted Gravelly Material" to be placed downstream of the original dam as a blanket drain.
3. The riprap removed from the upstream slope of the original dam to be placed at the downstream toe of the new dam in the vicinity of the outlet and gate chamber structure.

The materials referred to under 1 and 2 were not specified in the contract documents, but were obtained by field selection of the more pervious soils from the borrow area. Thus, it is not known how effective the zoning shown on the plans actually is in the field. Three holes were made with a hand auger to depths of 1.5 to 2.0 feet near the right catch basin in the upper berm of the downstream slope. Uncovered were about one foot of topsoil and then a gray clayey sand or sandy clay, which is too impervious to act as a drain.

c. Operating Records - The operating records available do not contain indications of instability.

d. Post-Construction Changes - The available records do not indicate changes after the 1958-1959 raising of the dam.

e. Seismic Stability - Lake Chamberlain Dam is located in Seismic Zone 1, according to the USCE recommended guidelines, and therefore, it does not require a special analysis for seismic stability.

f. Special Considerations - Seepage - If an analysis is made to determine the exit point of the line of seepage, the following may be concluded:

1) Assuming the dam to be homogenous, i.e. there is not an effective blanket drain, and assuming different ratios of horizontal, k_h, to vertical, k_v, permeabilities, the line of seepage will exit along the downstream slope at the following elevations for the maximum cross section.

   k_h/k_v = 1, Elev 352

   = 10, Elev 362

   = 100, Elev 382

2) Assuming the blanket drain to be effective, the seepage line will remain within the body of the dam for k_h/k_v equal to 1 and 10, and it will exit along the downstream slope if k_h/k_v = 100. The elevation at which the seepage line will exit will depend upon the degree of effectiveness of the blanket drain.

Ratios of horizontal permeability of 10 to 100 can be considered reasonable for embankments built in layers. On the basis of the analysis, it is probable that in some areas the line of seepage is discharging along the downstream slope whether the blanket drain is effective or not. The absence of wet spots on the downstream slope indicates the flow to be small enough so that evaporation and absorption by vegetation prevents the formation of wet areas. The presence of wet areas at the downstream toe, to the right of the outlet channel and also downstream of the dam, probably indicated that the observed seeps occur along the foundation soils and/or bedrock.

Neither the possibility of discharge along the downstream slope, nor the seeps observed, constitute an indication of an unsafe condition at the present time. However, the downstream slope of the dam is relatively steep, 2 horizontal to 1 vertical, and the internal drainage provisions are at best of limited effectiveness. Therefore, the effects of water discharging through the toe and downstream of the toe should be further investigated, and if necessary, seepage control measures such as toe drains or weighted filters should be installed.

7.1 Dam Assessment

a. Condition - Based upon visual inspections at the site and the past performance of the dam, the dam appears to be in good condition. No evidence of structural instability was observed, and the condition of the (earthen) embankment is good. However, there are some areas requiring monitoring and minor maintenance.

Based upon our hydraulics computations, the spillway capacity is 8100 cubic feet per second (cfs), which is in excess of 100 percent of the Test Flood. Based upon "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March 1978, peak inflow to the reservoir is 7600 cfs; peak outflow (Test Flood) is 5500 cfs. The spillway will pass 100% of the 5500 cfs test flood with the dam maintaining a 2.7 foot freeboard.

Utilizing the April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam would be 251,000 cfs. The average stage 5600 feet downstream to Glen Lake would be 34 feet. Glen Lake Dam would be overtopped by approximately 20 feet and would most likely breach. Even if the Glen Lake Dam does not breach, the 20 foot wave would sweep down the Sargent River to residential Woodbridge, approximately 1 mile further downstream, causing severe damage to life and property.

b. Adequacy of Information - The information available is such that an assessment of the stability of the dam must be based principally on visual inspection and past performance of the structure. For example, information concerning the "as built" zoning of the dam, which was not available, is essential to formally analyze the stability of an earth dam.

c. Urgency - The recommendations and remedial measures presented in Sections 7.2 and 7.3 should be implemented within the time span specified for each section.

d. Need for Additional Information - The findings of the visual inspection do not require further studies; however, the owner should perform additional investigations and monitoring as recommended below in Sections 7.2 and 7.3.

7.2 Recommendations

The recommendations presented in this section should be implemented within one year of the owner's receipt of this Phase I Inspection Report.

1. An investigation of the seep which exits downstream of the dam should be conducted to determine:

   a. Location of the exit points which are now obscured by vegetation.

   b. Potential for piping or boils at the location of the exit points (which depends on the type of soil at those points).

   c. Whether seepage control measures are indicated.

2. A program for monthly monitoring of seeps observed at the toe and downstream of the dam should be implemented. Monitoring should be visual to evaluate the turbidity of the water and should also include photographic evidence that would provide a record to detect large changes in the volume of flow or in the size of the wet areas from the time of one inspection to another. Presence of suspended solids in the water or substantial changes in flow not related to changes in reservoir level should be considered as indications of an unsafe conditions.

7.3 Remedial Measures

a. Alternatives - This study has identified no practical alternatives to the above recommendations.

b. Operation and Maintenance Procedures - The following measures should be undertaken within one year of the owner's receipt of this report and continued on a regular basis.

1. A formal program of operation and maintenance procedures should be instituted, and fully documented to provide accurate records for future reference.

2. Round the clock surveillance should be provided by the owner during periods of unusually heavy precipitation. The owner should develop a formal warning system with local officials for alerting downstream residents in case of emergency.

3. The spillway channel wall on the left side at the construction joint should be observed periodically to determine whether or not further movement is occurring.

4. During the course of this study, it was brought to our attention that the New Haven Water Company has instituted a yearly program for inspection of all their dams, including Lake Chamberlain Dam, by a consultant competent in the field of dam inspection. This program, which has been in effect for the past two years, is commendable and should be continued in the future.

APPENDIX

SECTION A: VISUAL OBSERVATIONS

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Lake Chamberlain

DATE: June 1, 1978

TIME: ——

WEATHER: Sunny, Clear

W.S. ELEV. 395 **U.S.** —— **DN.S**

PARTY:	INITIALS:	DISCIPLINE:
1. Mike Horton	MH	Structural
2. Hector Moreno	HM	Hydraulic
3. Gonzalo Castro	GC	Geotechnical
4. Dean Thomasson	DT	Party Chief
5.		
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Zoned Earth Dam Embankment	GC	
2. Spillway-Approach, Channel, Weir, Discharge Channel	GC/MH	
3. Outlet Works-Outlet Structure and Outlet Channel	GC	
4. Outlet Works-Service Bridge (Pedestrain/Vehicular)	MH	
5. Outlet Works-Control Tower, Operating House, Gate Shafts	MH	
6. Reservoir	DT	
7. Operation and Maintenance	DT	
8. Safety and Performance Instrumentation	DT	
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain **DATE** June 1, 1978

PROJECT FEATURE Zoned Earth Dam Embankment

AREA EVALUATED	BY	CONDITION
Crest Elevation		
Current Pool Elevation		
Maximum Impoundment to Date		
Surface Cracks	GC	None observed.
Pavement Condition	GC	Not applicable.
Movement or Settlement of Crest	GC	None apparent.
Lateral Movement	GC	None apparent.
/ertical Alignment	GC	No misalignment observable.
Iorizontal Alignment	GC	No misalignment observable.
Condition at Abutment and at Concrete Structures	GC	Good.
Indications of Movement of Structural Items on Slopes	GC	None apparent.
Trespassing on Slopes	GC	Motorcycle paths on D.S. slope.
Sloughing or Erosion of Slopes or Abutments	GC	None observed.
Rock Slope Protection-Riprap Failures	GC	None observed.
nusual Movement or Cracking at or near Toes	GC	None observed.
nusual Embankment of Downstream Seepage	GC	Significant seeps at toe and D.S. of Dam.
iping or Boils	GC	None observed.
oundation Drainage Features	GC	None known or observed.
oe Drains	GC	Along D.S. slope to the right of outlet structure.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain **DATE** June 1, 1978

PROJECT FEATURE Zoned Earth Dam Embankment

AREA EVALUATED	BY	CONDITION
Instrumentation Systems		
Vegetation	GC	D.S. slope grass covered.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain DATE June 1, 1978

PROJECT FEATURE Spillway-Approach, Channel, Weir, Discharge Channel

AREA EVALUATED	BY	CONDITION
a. Approach Channel		
General Condition		
Loose Rock Overhanging Channel		
Trees Overhanging Channel		
Floor of Approach Channel		
b. Weir and Training or Sidewalls		
General Condition of Concrete	MH	Good.
Rust or Staining		
Spalling	MH	Some at joints.
Any Visible Reinforcing	MH	None.
Any Seepage or Efflorescence	MH	Yes.
Drain Holes	GC	Good condition.
c. Discharge Channel		
General Condition	GC/MH	Good.
Loose Rock Overhanging Channel	GC/MH	None observed.
Trees Overhanging Channel	GC/MH	None observed.
Floor of Channel	GC/MH	Good, bedrock.
Other Obstructions	GC/MH	None observed.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain **DATE** June 1, 1978

PROJECT FEATURE Outlet Works-Control Tower, Operating House, Gate Shafts

AREA EVALUATED	BY	CONDITION
a. Concrete and Structural		
General Condition	MH	Good.
Condition of Joints	MH	Good.
Spalling	MH	Very little. Top surface spalled in some areas.
Visible Reinforcing	MH	None.
Rusting or Staining of Concrete	MH	None.
Any Seepage or Efflorescence	MH	Occasional.
Joint Alignment	MH	Good.
Unusual Seepage or Leaks in Gate Chamber	MH	None visible. Water in chamber spraying from inlet piping.
Cracks		
Rusting or Corrosion of Steel		
b. Mechanical and Electrical		
Air Vents		
Float Wells		
Crane Hoist		
Elevator		
Hydraulic System		
Service Gates		
Emergency Gates		
Lighting Protection System		
Emergency Power System		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain **DATE** June 1, 1978

PROJECT FEATURE Outlet Works-Outlet Structure and Outlet Channel

AREA EVALUATED	BY	CONDITION
General Condition of Concrete		
Rust or Staining		
Spalling		
Erosion or Cavitation		
Visible Reinforcing		
Any Seepage or Efflorescence		
Condition at Joints		
Drain Holes	GC	None observed.
Channel	GC	Natural river stream.
Loose Rock or Trees Overhanging Channel	GC	None of any significance.
Condition of Discharge Channel	GC	Good.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain **DATE** June 1, 1978

PROJECT FEATURE Outlet Works-Service Bridge (Pedestrian/Vehicular)

AREA EVALUATED	BY	CONDITION
a. Super Structure		
Bearings	MH	Acceptable.
Anchor Bolts		
Bridge Seat	MH	Good.
Longitudinal Members	MH	Good.
Under Side of Deck	MH	Good.
Secondary Bracing	MH	None.
Deck	MH	Pitched-good.
Drainage System		
Railings	MH	Spalling at base anchors.
Expansion Joints	MH	Joint filled with mortar; cannot close.
Paint	MH	None.
b. Abutment & Piers		
General Condition of Concrete	MH	Good.
Alignment of Abutment	MH	Acceptable.
Approach to Bridge	MH	Good.
Condition of Seat & Backwall	MH	Good.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain Dam **DATE** June 1, 1978

PROJECT FEATURE Reservior

AREA EVALUATED	BY	CONDITION
Shoreline	DT	Good.
Sedimentation	DT	None observed.
Potential Upstream Hazard Areas	DT	Closest house 1000'. No flooding potential.
Watershed Alteration-Runoff Potential	DT	None at this time.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain Dam **DATE** June 1, 1978

PROJECT FEATURE Operations and Maintenance

AREA EVALUATED	BY	CONDITION
Reservoir Regulation Plan		
Normal Conditions	DT	Daily water level readings taken.
Emergency Plans	DT	Report emergencies to the New Haven Water Company office.
Warning System	DT	
Maintenance (Type) (Regularity)		
Dam	DT	Maintenance when needed is reported to office. Maintenance and greasing usually every one (1) to two (2) years.
Spillway	DT	
Outlet Works	DT	Seepage since dam was built. Situtation presently being investigated and solutions considered.

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Chamberlain Dam **DATE** June 1, 1978

PROJECT FEATURE Safety and Performance Instrumentation

AREA EVALUATED	BY	CONDITION
Headwater and Tailwater Gages	DT	Measured at spillway.
Horizontal and Vertical Alignment Instrumentation (Concrete Structures)	DT	None.
Horizontal and Vertical Movement, Consolidation, and Pore-Water Pressure Instrumentation (Embankment Structures)	DT	None.
Uplift Instrumentation	DT	None.
Drainage System Instrumentation	DT	None.
Seismic Instrumentation	DT	None.

APPENDIX
SECTION B: EXISTING DATA

SPECIAL NOTE

SECTION B

AVAILABILITY OF DATA

The correspondence listed in the Summary of Contents and the plans listed in the Table of Contents, Appendix Section B, are included in the master copy of this report, which is on file at the office of the Army Corps of Engineers, New England Division, in Waltham, Massachusetts.

Only the following correspondence is included in this report:

Date	To	From	Subject	Page
July, 1958	New Haven Water Co.	Malcolm Pirnie Engineers	Design Report Chamberlain Lake Dam	B-3
June 26 1965	New Haven Water Co.	Joseph W. Cone	Report concerning dams owned by New Haven Water Co.	B-102
Aug. 2, 1967	New Haven Water Co.	Malcolm Pirnie Engineers	Investigation of the effects of a flood produced by the Maximum Possible Storm on spillways of West River System	B-130

SUMMARY OF CONTENTS

DATE	TO	FROM	SUBJECT	PAGE
April 29, 1963	A.L. Corbin, Jr.	Joseph A. Navaro, Chief Engineer, New Haven Water Company[2]	West River Watershed	B-87
April 12, 1965	Joseph W. Cone	New Haven Water Company[2]	Transmittal of (and including) Chamberlain Data Form	B-90
April 30, 1965	Joseph W. Cone	New Haven Water Company[2]	Transmittal of (and including) Lake Level and	B-91
June 26, 1965	New Haven Water Company	Joseph W. Cone[2]	Report Concerning Dams Owned By New Haven Water Company 3	B-101
July 24, 1965	William Sander	Joseph W. Cone[2]	Corrections on Report Concerning Dams Owned by New Haven Water Company	B-123
July 15, 1966	William Wise, Water Resources Commission	Joseph Novaro, New Haven Water Company[1]	Progress Report for West River System Studies	B-129
Aug. 2, 1967	New Haven Water Company	Malcolm Pirnie Engineers[1]	Investigation of the Effects of a Flood Produced by the Maximum Possible Storm on Spillways of West River System	B-130
Original Date Mar. 1, 1911; Latest Entry 1969	New Haven Water Company	Albert B. Hill[2]	Reservoir Capacities, West River System	B-147

DATE	TO	FROM	SUBJECT	PAGE
Aug. 1974	Files	New Haven Water Company[2]	Chamberlain Dam Data Sheet & Photographs	B-149

[1]Obtained from State of Connecticut Water Resources Commission

[2]Obtained from New Haven Water Company

[3]Hydraulic/Hydrologic Data and Spillway Sections contained in Joseph W. Cone's report, which are on file and available at the New Haven Water Company office were not included due to poor reproduction quality.

NEW HAVEN WATER COMPANY

NEW HAVEN, CONNECTICUT

DESIGN REPORT

CHAMBERLAIN LAKE DAM

JULY 1958

MALCOLM PIRNIE ENGINEERS
25 West 43rd Street
New York 36, N.Y.

NEW HAVEN WATER COMPANY

NEW HAVEN, CONNECTICUT

DESIGN REPORT

CHAMBERLAIN LAKE DAM

<u>LOCATION</u>

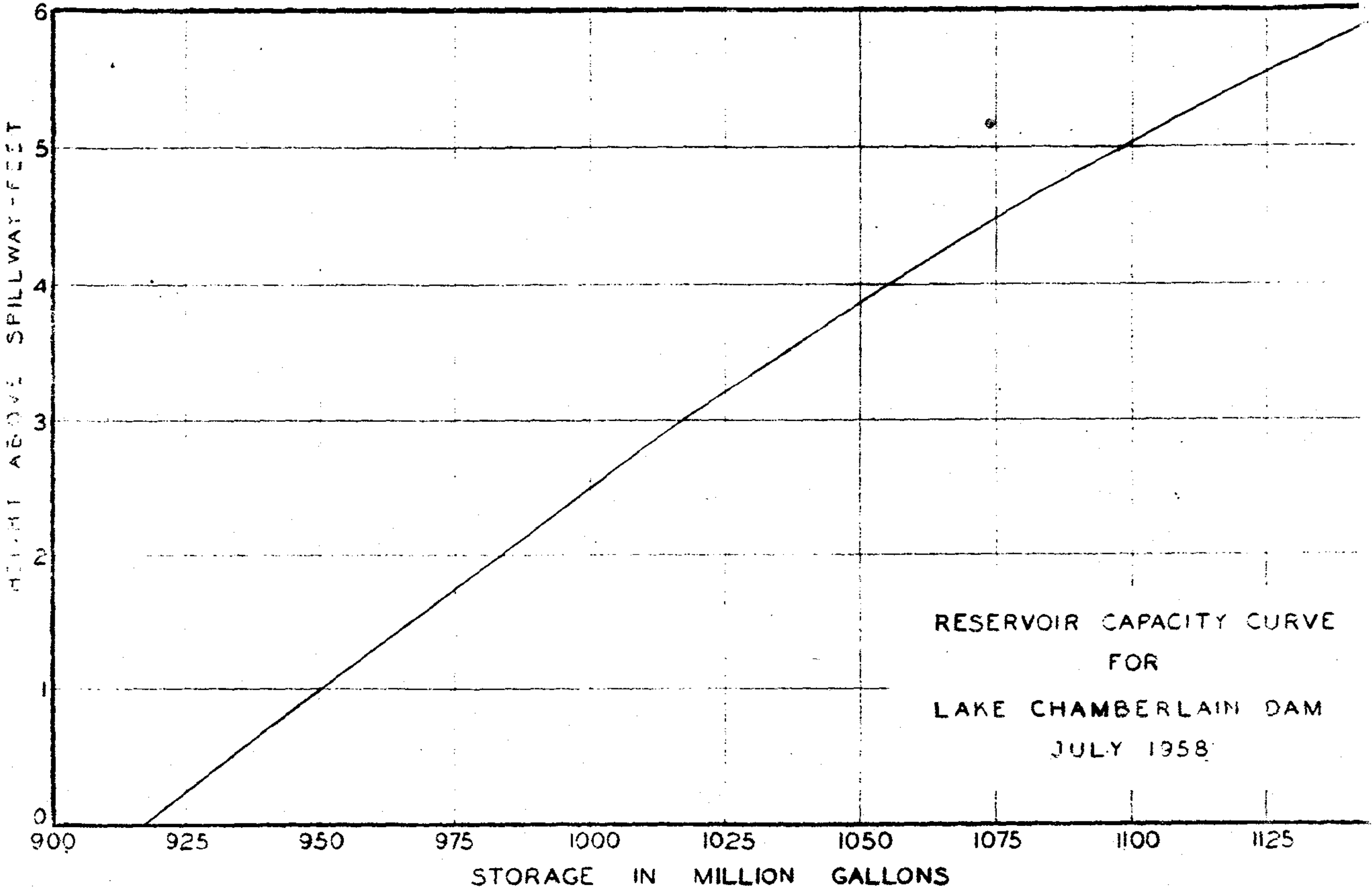
The Chamberlain Lake Dam is located on Sargent River about 4,000 feet west of Route 69 in Bethany. The location is shown on Sheet 1 of 8 of the contract drawings.

<u>DESCRIPTION OF PROJECT</u>

At present the New Haven Water Company has two reservoirs on the Sargent River. Glen Lake, with a usable storage of 197 million gallons has a tributary drainage area including that of Chamberlain Dam of 5.6 square miles. Chamerlain Lake, with a usable capacity of 165 mg, is located upstream from Glen Lake and has a tributary drainage area of 3.9 square miles.

It is proposed to raise the Chamberlain Lake Dam spillway from Elevation 359.87 to Elevation 395.0, increasing the usable storage to 900 mg. The safe yield of the Glen Lake-Chamberlain Lake system will be increased from 2.9 mgd to 4.7 mgd. The capacity curve of the proposed reservoir is shown in Figure 1.

The present Lake Chamberlain Dam, constructed in 1891, is a rolled earth dam with a masonry core wall. The entire


HEIGHT ABOVE SPILLWAY - FEET
0
1
2
3
4
5
6
900
925
950
975
1000
1025
1050
1075
1100
1125
STORAGE IN MILLION GALLONS
RESERVOIR CAPACITY CURVE
FOR
LAKE CHAMBERLAIN DAM
JULY 1958

dam is built on rock with a 39-foot long spillway cut in the rock at the easterly end of the dam. There are two 30-inch cast iron blowoffs through the dam which are used to let water down to Glen Lake.

The arrangement of the higher dam will be very similar to the existing one. The dam will be a compacted earth structure. The two 30-inch blowoff lines will be used with a new intake and outlet structure. The spillway will be a 50-foot concrete ogee section cut into rock at the easterly end of the dam.

FLOOD FLOWS

There is no record of stream flow gagings on the Sargent River.

Peak flows at the dam site have been estimated by the procedure outlined in Geological Survey Circular 365 as modified in the Connecticut Society of Civil Engineers' 73rd Annual Report, Pages 89 and 92. Peak floods and flood hydrographs were calculated according to procedures outlined in the Army Corps of Engineers' Engineering Manual, Hydrologic and Hydraulic Analyses, Part CXIV, Chapter 5.

Judging the drainage area to have normal characteristics in accordance with Circular 365 nomenclature, the peak flood with a recurrence interval of 1,000 years was estimated to be 2,940 cfs.

Using rainfall data from the U. S. Weather Bureau Technical Paper No. 25, Rainfall Intensity-Duration-Frequency Curves for the New Haven Weather Station and that at Meriden, Connecticut, and Westfield, Massachusetts for the hurricane storms of August, 1955, a storm of hurricane intensity was developed with a recurrence interval of once in 1,000 years.

A flood hydrograph constructed by the Army Corps of Engineers' method is shown in Figure 2. The peak flow is 3,055 cfs as compared to 2,940 cfs by the method used in Circular 365.

FLOOD ROUTING

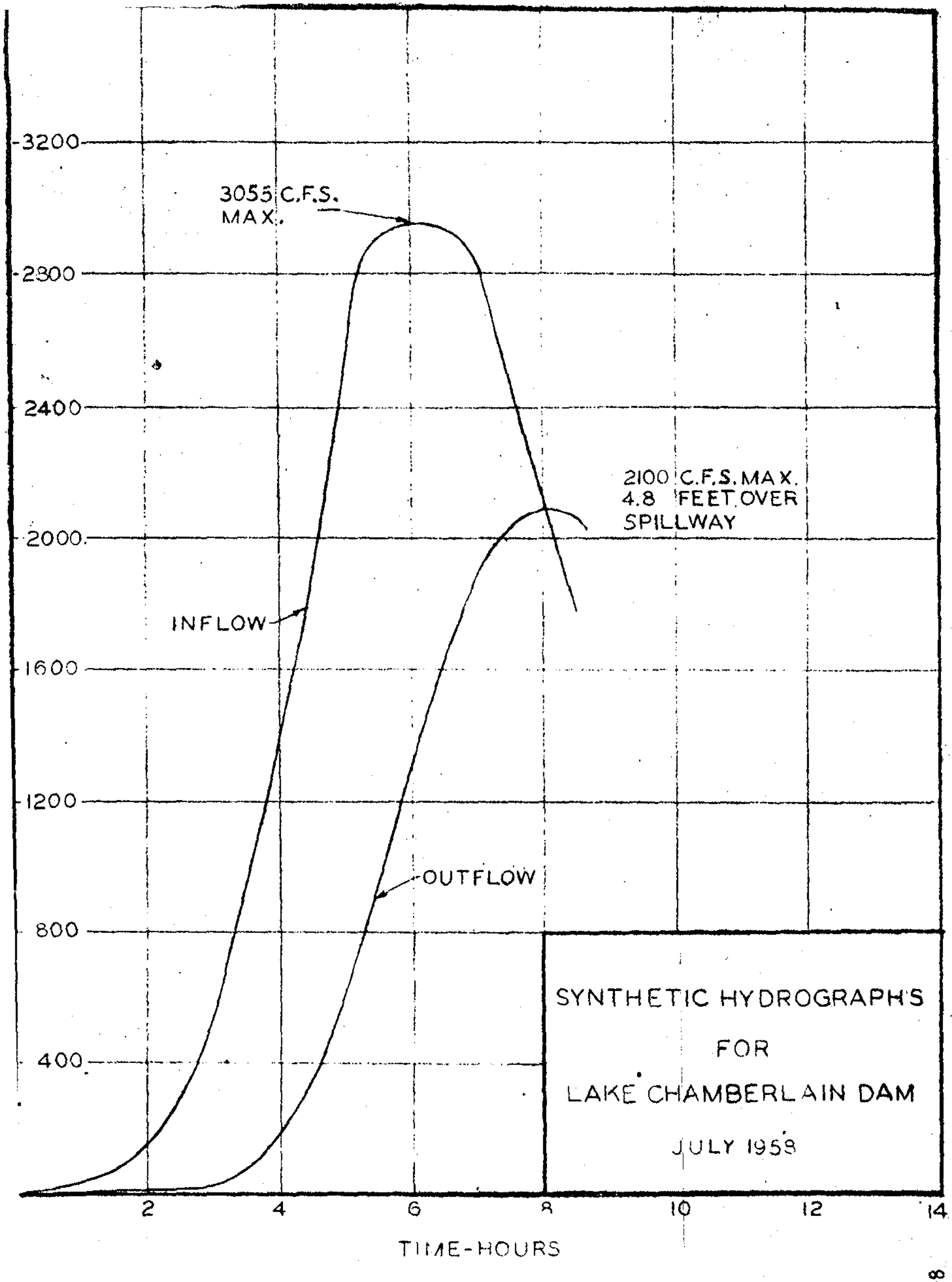
The flood hydrograph for a 1,000-year storm developed by the Army Corps of Engineers' method was routed through Lake Chamberlain. A spillway rating curve, shown in Figure 3, was used for the ogee type spillway section with a length of 50 feet and a coefficient which varies with the head on the spillway. The coefficient varies from 3.2 to 3.9.

When the design storm was routed through the reservoir a peak outflow of 2,100 cfs and a head of 4.8 feet above the spillway crest was obtained. This is indicated in Figure 2. Freeboard of 7 feet above the maximum water level resulting from a 1,000-year flood will enable the dam to pass floods of a recurrence interval considerably greater than once in 1,000 years with no damage other than possible local damage to the spillway outlet channel.

The outlet channel has been designed to handle a peak outflow of 2,100 cfs.

PLANS AND SPECIFICATIONS

The contract drawings consist of eight (8) sheets which show plans, sections, elevations and details of gate chamber,

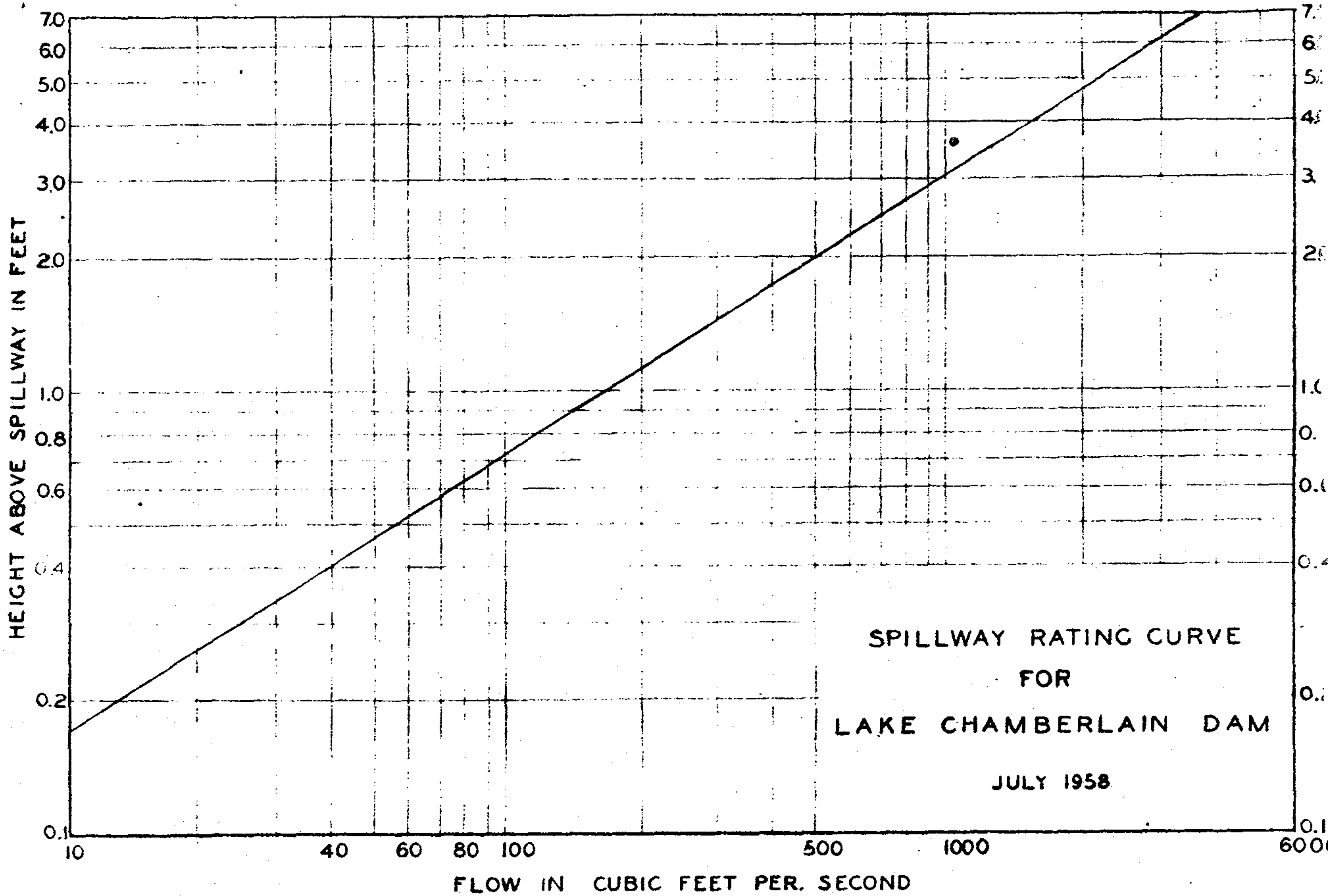


SYNTHETIC HYDROGRAPH'S

FOR

LAKE CHAMBERLAIN DAM

JULY 1958


HEIGHT ABOVE SPILLWAY IN FEET
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.8
0.6
0.4
0.2
0.1
10
40
60
80
100
500
1000
6000
FLOW IN CUBIC FEET PER. SECOND
SPILLWAY RATING CURVE
FOR
LAKE CHAMBERLAIN DAM
JULY 1958

concrete spillway and overflow structures. The dam will be a compacted earth dam. Suitable material exists in the reservoir area immediately above the dam.

It is planned to have full-time engineering supervision during the construction of the dam, control of moisture content, degree of compaction and density of compacted embankment will be maintained.

MALCOLM PIRNIE ENGINEERS

1965

REPORT

CONCERNING DAMS

Owned by

NEW HAVEN WATER CO.

BETHANY

WATROUS

CHAMBERLAIN

GLEN

DAWSON

on the

WEST & SARGENT RIVERS

J. W. Cone P.E.
June 1965

B-102

INDEX

Part I

Part II

NOTE: Maps, graphs, etc., are in separate folder.

LICENSE 4755
IT REGISTRATION 4

JOSEPH W. CONE
CIVIL ENGINEER
124 HAVEMEYER PLACE
GREENWICH, CONNECTICUT
06830

TELEPHONE
TOWNSEND 9-2152

June 26, 1965

Mr. William P. Sander
Water Resources Commission
State Office Building
Hartford 15, Conn.

Re: Dams #35 - 1 to 5
New Haven Water Co.

Dear Mr. Sander:

First, I apologize for not completing this assignment more promptly; reasons being that a low quality virus for over a month left me with no pep mentally or physically, and delays in obtaining certain plans and information.

The assignment was- "we would like to know the present condition of these dams" - Bethany - Watrous - Dawson on West River and Chamberlain - Glen on Sargent River, a tributary to West River above Dawson Dam.

In my opinion, the "condition" of these dams is good as regards masonry of the three masonry gravity dams and the upkeep of two earth embankment dams.

But as regard to whether or not the dams are safe, <u>particularly as regard spillway capacity</u>, my opinion is as follows:

35-1 <u>Bethany</u> Spillway is inadequate. However a thin sheet over a length of 990' will do comparatively little damage except to highway. The gravity section is safe.

B-104

1

35-2 <u>Watrous</u> Generally same remarks as for Bethany.

35-3 <u>Chamberlain</u> Spillway is adequate in every respect as is the dam. It is reassuring to find a spillway that will carry 1525 cfs per sq. mi. on 4.1 sq. mi. Note Items #26 & 28 on Data Sheet.

35-4 <u>Glen</u> Spillway is nowhere near adequate. In fact, Oct. '55 flood nearly overtopped earth section at left or east abutment. Section of dam is safe.

Right abutment should be raised to protect highway.

Left abutment should be investigated:-

(a) To determine whether or not there is a core wall.

(b) Possibility of emergency spillway or fuse plug.

(c) Note Items #26 & 28 on Data Sheet.

35-5 <u>Dawson</u> Present spillway is entirely inadequate to carry probable floods of the present and future. In fact, the dam would have been overtopped if certain saving factors had not been present in Oct. 1955.

(a) Not an excessive rainfall, only about R of 50 yr. (Compare with precipitation graphs)

(b) Several of reservoirs were below FL (See data notes by Navaro which you have)

(c) Flood Q '55 at Dawson of about 2100 cfs has an R value 3.8 (2100 ÷ 560) equivalent to 120 yr on old Conn. curve and 55 yr on revised 1965 curve. (See graph PL 13)

Items #26 & 28 on Data Sheet are particularly illuminating.

It does not need a lively imagination to visualize what would happen to Westville and New Haven if Dawson should be overtopped; Norwich failure would be peanuts comparatively.

A brief discussion of pertinent data and situations follows. Also there are prints of sections of dams, precipitation graphs and various other graphs that I used or are pertinent to this investigation for general information or checking purposes.

Please excuse the informality and crudness of the matter submitted, the objective being to reduce costs to the minimum.

I would observe that Mr. Navaro, Mr. Ferris and Mr. Reynolds of the New Haven Water Co. were most cooperative as was Mr. Thomas of the U.S. Geological Survey.

My recommendation is that the New Haven Water Co. be advised that their consulting engineers should investigate the entire system, with particular emphasis on conditions at Glen and Dawson, and submit corrective measures.

Yours very truly,

J W Cone

J. W. Cone

JWC/dr

Enc: Part II
Photos (11)

B-107

4

<u>Characteristics</u> Area is very rugged, steep side slopes and steep channels. Channel slopes (S in Conn Formula) are West River 70 and Sargent River 88 feet per mile. Elevations on topo sheet point up steepness of side slopes as much as 400' in 0.25 mile.

Area is rural, cover, mostly wooded at present. However within a few decades there will be more intensive land use. There is evidence of this growth in the Cheshire and other areas. At present in spite of rugged terrain, the shed may be considered "medium to fast" due to cover; by about 2000 AD it will become "fast" and in the future could be "very fast".

<u>Area</u> As scaled from 1:24,000 topo sheets area is 13.35 sq. mi. By data in Water Co's. operation office area is 13.0 sq. mi. Mr. Novaro in his report to Mr. Corbin, April 29, 1963, states area is 13.9 sq. mi.; this I do not understand.

	Water Co.	1:24000
Bethany	3.4	3.7
Watrous	3.2	3.3
Chamberlain	3.9	4.1
Glen	1.7	1.6
Dawson	.8	.65
	13.0	13.35

The Company owns about 8 sq. mi. of the 13.35 sq. mi. However as taxes and population pressures increase, as the area becomes more polluted due to

B-108

development of areas owned by others, it is reasonable to assume that the Company will sell at least 5 sq. mi. and construct a filtration plant. These considerations explain the predicted increase in mean annual flood of about 40% above present by 2000 AD. (560-795 and C_B 0.85-1.2)

The following quote, from an intensive study by Metcalf and Eddy on Storm Water Control in Westchester County in 1945, is pertinent to this discussion. "Residential development of the area has resulted in peak run-off rates almost twice those of twenty-five or thirty years ago, and if development continues at the same rate for the next twenty-five years, the run-off factor will become 2½ times that of conditions a half century ago". It would seem that the increase of 40% is not fantastic.

PRECIPITATION

Data plates 4 to 9 inclusive were studied and are included to determine whether or not the Oct. 1955 storm in the New Haven area was of very rare occurence.

Since the rain gage at Dawson is not recording, graph PL 5 was produced assuming that storm characteristics would be very similar to New Haven Airport which has a recording gage. Similarly the Westfield, Mass. graph was based on Norfolk, Conn.

B-109

6

Using 24 hr values and PL 9 the following recurrence values were determined.

	24 hr in.	Chance %	R
Base	9.5	1.0	100
Dawson	5.85	2.0	50
Norfolk	11.2	0.6	175
Westfield	18.2	0.2	500
Max possible	27.7	0.1	1000

In connection with this subject on Oct. 9, 1877 there was 9.7" in 10.5 hrs. at White Plains, Westchester County, N.Y.

My conclusion is that precipitation in the New Haven area cannot be termed extraordinary. In the Stamford-Norwalk area R values were about 200 yr and in Greenwich about 75.

If precipitation was not excessive then peak flood flow could not be excessive and should have an R value of less than 100.

I realize full well that some may say that I have no right to assign maximum possible to 1000 yrs. My answer is what possible value can the maximum possible values have unless an occurrence value is stated; if no value then data is worthless. Enquiry has been made to many who should be better versed in this matter than I. No one would stick his neck out. I am not afraid to and have; at least a value of 1000 is on the safe side.

My purpose in this discussion is to point out the fact that if either the Norfolk or Westfield precipitations had occurred on this shed in Oct. '55 the resulting disaster would have been appaling.

FLOOD FLOW 1955

Oct. 1955 To determine flood flow at Dawson it is necessary to know H at peak. To check, if H at peak were known for Glen and Watrous, then flow to Dawson could be estimated reasonably close by adding an allowance for the small watershed of Dawson itself.

In this connection I suggest that values shown on Lake Level forms (those were mailed you recently) should not be used since measurements were taken between 8-9 A.M.

The peak of the Oct. flood in Greenwich was about 1 A.M. Allowing for forward speed of storm then peak at Dawson would be between 2-3 A.M. particularly since watershed is "quick". The time lag of about 6 hours would certainly lower H peaks. I therefore, based on conversations and data furnished, assumed certain H values and computed Q, as shown in the following table:

	H	Q	
Glen	3.5	880	cfs
Watrous	3.0	1160	
Dawson shed est		160	
		2200	" to check
Dawson	4	2050	"

B-111

Assuming 2050 correct than R values are:

$$R = \frac{Q}{MAF} = \frac{2050}{560} = 3.7 \pm$$

Refer to PL 13

		R
By old Conn Curve	3.7	110 yrs
" new " "	3.7	50 "

This agrees reasonably well with precipitation value of 50.

Conclusion is that flow of Oct. 1955 at Dawson may be considered a minor flood that would have been somewhat greater had not several of the reservoirs been below FL. for a total of 215 m.g. as computed by Mr. Novaro.

$Q_M = 9\ A^{2/3}$ vs Conn Formula

This formula and graph (PL 12 A & B) has been used for several years with satisfaction. It checks well with the rational method and is much simplier to use. Although designed for small watersheds, up to about two square miles, it fills the gap with considerable reliability up to about ten miles, the approximate reliable lower limit of the Conn Formula, Geological Survey Circular #365.

A = 13.35 sq. mi. = 8500 Ac

8500	3.92942
	2
	3 / 7.85884
	2.61961
9	0.95424
3748	3.57385

Q_M = 3750 cfs

8-112

9

$\underline{Q_D = RF \times LF \times FF \times Q_M}$

From PL 12 A factors for R = 500, present conditions and 2000 AD Present Q = 1 x 0.4 x 4.35 x 3750 = 6500

2000 AD 1 x 0.6 x 4.35 x 3750 = 9730 $\frac{0.6}{0.4} = 1.50$

$Q = RC_B\,AS$

By PL #2 C_BAS = 560-present and 795-2000 AD

By PL 13 R for 500 = ~~11~~ 12

Q500 Present = 11 x 560 = ~~6160~~ 6720

" 2000 AD = 11 x 795 = ~~8745~~ 9540

Note that results are remarkably close, perhaps by coincidence.

	$9\ A^{2/3}$	C_B		Conn	C_B	
Present	6500	0.4	150%	6160	0.85	140%
2000 AD	9730	0.6		8745	1.2	

Had basin coefficients (C_B) been selected to obtain the same percent increase in the land use factor, results for 2000 AD would have been 9730 vs ~~9240~~. 10,080

In any case $Q = 9\ A^{2/3}$ provides a reliable check on Conn. Formula, up to about 10 sq. mi., and fills the no-man's gap.

B-113

SPILLWAY CAPACITY

cfs. & sq. ft. per sq. mi.

Dam	Type	Q / sq.mi.	cfs	sq.ft.
(1) Bethany	Gravity	1980 / 3.7	540	80
(2) Watrous	"	2660 / 7	380	50 acc
(3) Chamberlain	Earth	6300 / 4.1	1525	120
(4) Glen	Gravity	1120 / 5.7	195	28 acc
(5) Dawson	Earth	2870 / 13.35	215	30 acc

The units shown in this table, for a watershed with nearly the same characteristics throughout, demonstrate the inconsistency in capacity. It is true that an earth dam should have a greater factor of safety than a gravity masonry dam. This data emphasizes the need for corrective measures particularly at Glen and Dawson.

MAF

Check by Comparison

	Est.	Sq. Mi.	Present MAF	per/S.M.
Willow Brook-Cheshire	1960	9.02	280	31
Wepawaug River-Milford	1962	18.00	690	38
		/27.00	/970	
		13.5	485	
Dawson computed PL.2		13.35	560	42
Sargent " "		5.7	425	75

WILLOW BROOK. Rolling terrain, nowhere near as rugged as West River. On other hand land use is more dense. MAF per sq. mi. should be much less than West River.

WEPAWAUG RIVER. Same remarks as above.

SARGENT RIVER. Very steep. S is 88' per mi.

Note that Willow Brook and Wepawaug River stations have only short term records. The usual experience is that the longer the record period the higher are MAF values.

CONCLUSION is that West River MAF of 560 for present land use conditions is not too high and more likely is too low.

(1) BETHANY

BRIDGE. Rough field measurements were taken believing that the bridge would be a bottleneck rather than the spillway. Sketch plan is shown. Later construction plans were available.

Assuming depth of flow in channel as 3' -

$$A = 24.5 \times 3 = 73.5$$

$$P = 24.5 + 6 = 30.5$$

$$r = 73.5 \div 30.5 = 2.4 \qquad r^{2/3} = 1.8$$

$$S = .034 \qquad S^{\frac{1}{2}} = 0.18$$

Assuming $n = .0148$

$$V = 100 \; r^{2/3} \; S^{1/2}$$

$$= 100 \times 1.8 \times 0.18 = 32 \text{ sf.}$$

$$Q = 73.5 \times 32 = 2350 \pm \text{ cfs.}$$

SPILLWAY. Rough plan shows total length of spillway as 19' + 61' = 80'. But account of turbulence assume effective L = 75', H max = 4', C = 3.3.

$$Q = 3.3 \times 75 \times 8 = 1980 \text{ cfs.}$$

This Q probably maximum due to backup from bridge and turbulence at channel entrance.

From the above it is shown that the spillway rather than the bridge is the limiting factor to carry estimated Q values - Items 14 & 15 on Data Sheet. It is concluded that the dam will be overtopped in the future, with an H value of about 1.

$$Q = 2 \times 990 \times 1^{3/2} = 2080 \text{ cfs}$$

This with spillway on H = 5' will pass over 4000 cfs.

DAM. The gravity section of cement rubble masonry with reinforced concrete back 4' thick is in good condition.

(2) WATROUS

SPILLWAY. The capacity of this 70' spillway with H = 5' is 2660 cfs., as shown by Item 12 on Data Sheet. This capacity will barely take flood flow from its individual watershed below Bethany under present land use, see Items 14 & 15. In addition there is the added flow from Bethany. Total watershed is 7 sq. mi.

Bethany	3.7
Watrous	3.3
	7.0

DAM. The gravity concrete section is in good condition and is backed up with earth nearly to top of dam.

The dam will be overtopped in the future. Note Data Items #26 & 28.

(3) CHAMBERLAIN

A study of items on the Data Sheet and examination of sketch plan indicate that this earth dam is adequate in every respect. No further comment is required.

(4) GLEN

<u>SPILLWAY</u>. The 40' x 4' spillway has a capacity of about 1120 cfs. The entire watershed including Chamberlain is 5.7 sq. mi. Note Data Items #26 & 28.

Chamberlain	4.1
Glen	1.6
	5.7

The dam was nearly over-topped during the October 1955 flood.

<u>ABUTMENTS</u>. A highway is close to the right or west end of spillway. Upstream training wall in particular should be raised and extended.

At the left or east end of the dam there is an area that is lower than crest of dam. This is indicated under the arrow on the photo of the east bank. As determined by hand level, the area is about six inches below dam crest.

There seems to be no record of a core wall in the area or location of ledge surface. If no wall and ledge rock is low, then there will be end scour sometime in the future that would put an extra burden on Dawson. This condition should be investigated.

FUSE-PLUG. The area appears to be favorable for the needed extra spillway capacity, permanent construction, or fuse-plug type.

DAM. The gravity concrete section is in good condition and in my opinion will not fail.

(5) DAWSON

SPILLWAY. An examination of Data Sheet items and study of plans indicate that the Dawson spillway is entirely inadequate. The Q of 2870 with H of 5' is approximate. The combination of a low broad crested humped weir and spillway characteristics present a complicated hydraulic problem not worthwhile to investigate thoroughly for the purpose of this report.

The spillway and right training wall are shown on photo enclosed. Note that the low portion of the training wall was nearly overtopped in Oct. '55.

Height of water at spillway was 3' below dam crest. There must have been considerable velocity head. Therefore if the weir formula is used H should be about 4'.

SEEPAGE. In the area near trees as shown on enclosed photo there is seepage with "guesstimated" flow of about 9 gals per min. Another seepage flow is farther to the west and at a lower elevation near a small cedar with an estimated flow of about 3 gals per min. Both areas should be watched closely.

It would be worthwhile to install a simple arrangement whereby flow can be determined by stop watch timing to fill a container; this to determine whether or not there is a relation between reservoir level and flow.

I have been informed by Mr. Ferris that most of the trees shown in photo have been removed. Trees were not on the embankment proper but were close enough to present the possibility of root-boil trouble.

EMBANKMENT COVER. The easterly portion of the dam, about one half, had been grazed by sheep. This is an inexpensive method of controlling grass on a 1 on 2 slope. On the other hand sheep are close croppers and tend to destroy root structure, a condition evident at the time. If the dam should be overtopped by a few inches I would anticipate that the sheep cropped area would gully seriously.

Further, particularly during dry weather, grass cover should be kept high to provide shade to hold moisture as much as is possible on the steep 1 on 2 slope, where water-table is low, and to prevent baking all of which weakens root structure.

CONCLUSION. It is my opinion that the situation at Dawson is very serious. If a bad breach should occur the refuge in "An Act of God" would not prevail. In Oct. 1955 if all reservoirs had been full, if twenty-four hour precipitation had been a little more, then it is my opinion that Dawson would have been overtopped.

As stated hereinbefore a comprehensive study of this situation should be begun immediately and proposed corrective measures presented as soon as possible.

GENERAL

It is my understanding that my assignment was not to undertake a complete analysis of all aspects involved, but only to investigate sufficiently to determine if there are situations that should be studied by the Company's consulting engineers. I therefore did not undertake the following:

1. Stability analysis of gravity masonry dams. Casual study of plans indicates they are safe; this based on experience.
2. A design flood based on an assumed precipitation was not routed through the several watersheds and reservoirs, considering storage capacity above FL etc. This would have been a tedious study and funds were not available in my contract.
3. In computing the several Q values no credit was given to storage above FL, rather this was considered as an extra factor of safety, to be on the safe side.

Graphs, plans, etc., are bound separately for ease in following the text.

DATA SHEETS

1. Summary of data.

2. Determination of MAF, graphically

3. Watersheds; sketch arrangement

4. Precipitation Oct. '55 New Haven

5. " " " Dawson (devised)

6. " Aug. " Norfolk

7. " " " Westfield (devised)

8. " Maximum Possible

9. " Recurrence 2 to 24 hr.

10. Flood flow graph old.

11. " " " revised.

12.- A Peak Runoff $Q = A^{2/3}$

B " " "

C " " "

13. Ratio Curve - Conn Formula

14. Weir Coefficients

15. Plans Bethany (3)

16. " Watrous (1)

17. " Chamberlain (1)

18. " Glen (2)

19. " Dawson (2)

Topo of Watershed 1:24000

COMMENTS re DATA SHEETS

#10 This Flood Flow Curve shown since it shows a curve, dashed line, devised by A.B. Hill about the turn of the century. It was considered a sound base curve at that time when there was a paucity of information as compared to that which became available in more recent years; precipitation and flood flow records, many studies, reports, etc.

#13 The upper curve, shown in red, was plotted by Mr. Mendall P. Thomas with the Geological Survey based on study by A. Rice Green, Water Supply Paper 1671, 1964. Curve has official approval to 100 years; projection to 1000 by Thomas using Gumbel's recurrence interval scale. This is the latest R-curve available.

The purpose of including the other sheets I believe is self-evident.

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT

MEMORANDUM REPORT TO WATER COMPANY
ON
INVESTIGATION OF THE EFFECTS OF A FLOOD
PRODUCED BY THE MAXIMUM POSSIBLE STORM
ON SPILLWAYS OF WEST RIVER SYSTEM

AUGUST 2, 1967

The effect of the "maximum possible storm" on the West River System is reported in this memorandum.

The "maximum possible storm" employed is defined and quantitatively estimated in U. S. Weather Bureau Hydrometeorological Report No. 33 entitled "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours." The report defines the "maximum possible precipitation" as "the critical depth-duration-area rainfall relation for a particular area during various months of the year that would result if conditions during an actual storm in the region were increased to represent the most critical meteorological conditions that are considered probable of occurrence."

As shown on Exhibit 1, the rainfall totals used for the West River System analyses are for durations of 6 and 12 hours on an area of 10 square miles for September -- the most severe month for the vicinity of New Haven, Connecticut. The hourly distribution of the total rainfall assumed is according to Figure 4, page 32 of U. S. Department of the Interior publication "Design of Small Dams." The distribution is a comparatively severe one with 50 per cent of the 6 hour total falling within 1 hour.

The sequence in which the hourly totals were arranged is in accordance with the recommendation made on page 50 in "Design of Small Dams." The arrangement of the 12 hourly increments is 11, 9, 7, 5, 3, 1, 2, 4, 6, 8, 10, 12, where the number represents the order of magnitude with the lowest number representing the largest magnitude. This arrangement gives a flood greater than one based on the assumption that the greatest hourly increment of rain occurs during the first hour of a storm

The effective, runoff-producing rainfall was estimated by subtracting 1 inch initial infiltration and 0.1 inch per hour thereafter from the total rainfall.

In order to pass the unusually high flows for the "maximum possible storm," several modifications of both the length and crest height of spillways were tried. Spillway rating curves and stage capacity curves for each of the five reservoirs are shown on Exhibit 2 and Exhibit 3, respectively.

The unit-hydrographs and routing procedures employed are those outlined in our report of January, 1967. Detailed computations are shown on Exhibit 4, pages 1 through 8.

The inflow-outflow curves for each of the reservoirs are shown on Exhibit 5, pages 1 though 3. As no significant storage effect is obtained from Lake Dawson, the outflow hydrograph as shown on Exhibit 5, page 3, will be the same with a spillway 250 feet long.

The "maximum possible" flood outflows at each of the West River reservoirs and the conditions at the Spillways are summarized below:

Dam	Peak Spillway Discharge cfs	Free-Board ft.	Maximum Head (ft.) Over Spillway	Over Dam Crest
Chamberlain	7200	12.0	10.8	-1.2
Glen	9665	9.0*	11.3	+2.3
Bethany	7350	4.25	5.2	+1.0
Watrous	15,400	5.0	7.1	+2.1
Dawson				
80' Spillway	26,260	11.5*	13.8	+2.3
250' Spillway	26,260	11.0*	9.0	-2.0

*Freeboard above proposed new sill elevation

MAXIMUM POSSIBLE RAINFALL FOR NEW HAVEN, CONNECTICUT

*DURATION OF RAINFALL HOURS	TOTAL RAINFALL INCHES
6	24.2
12	26.4

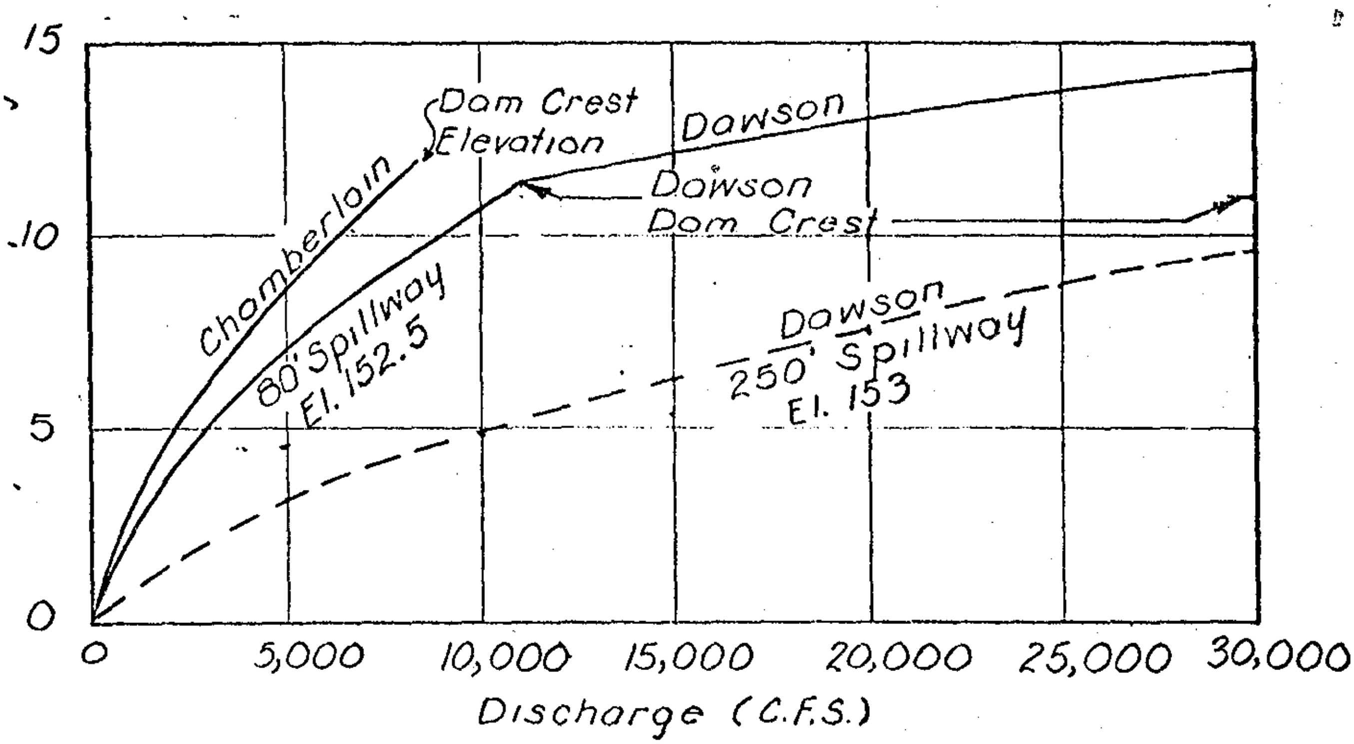
DISTRIBUTION OF 6 AND 12 HR. TOTALS

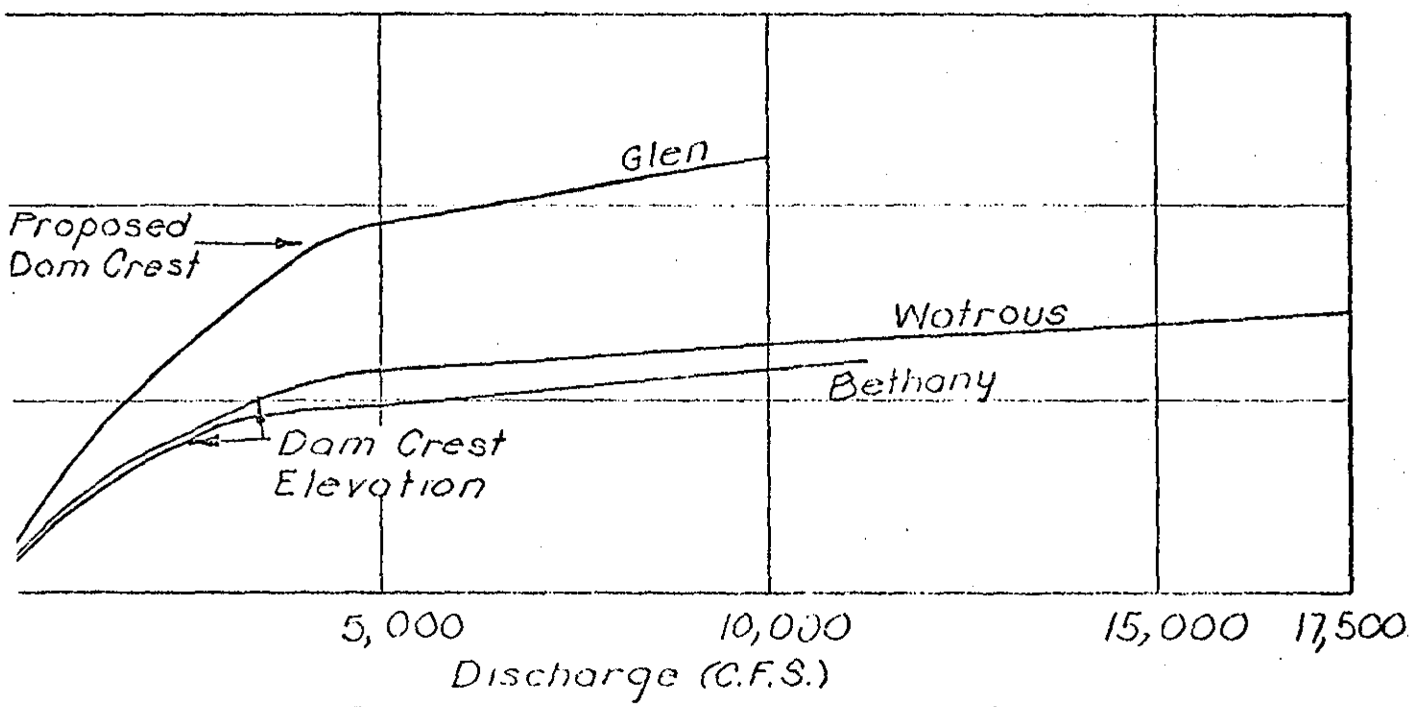
TIME FROM BEGINNING OF RAIN HOURS	**INCREMENTAL RAINFALL INCHES	** REARRANGED	LESS 1" INITIAL & 0.1" INFILTRATION PER HOUR
1	12.1	0.1	--
2	3.6	0.3	--
3	2.6	1.0	0.3
4	2.2	1.9	1.8
5	1.9	2.6	2.5
6	1.8	12.1	12.0
7	1.0	3.6	3.5
8	0.5	2.2	2.1
9	0.3	1.8	1.7
10	0.2	0.5	0.4
11	0.1	0.2	0.1
12	0.1	0.1	--
	26.4	26.4	24.4

*From Weather Bureau Technical Paper 33 1956

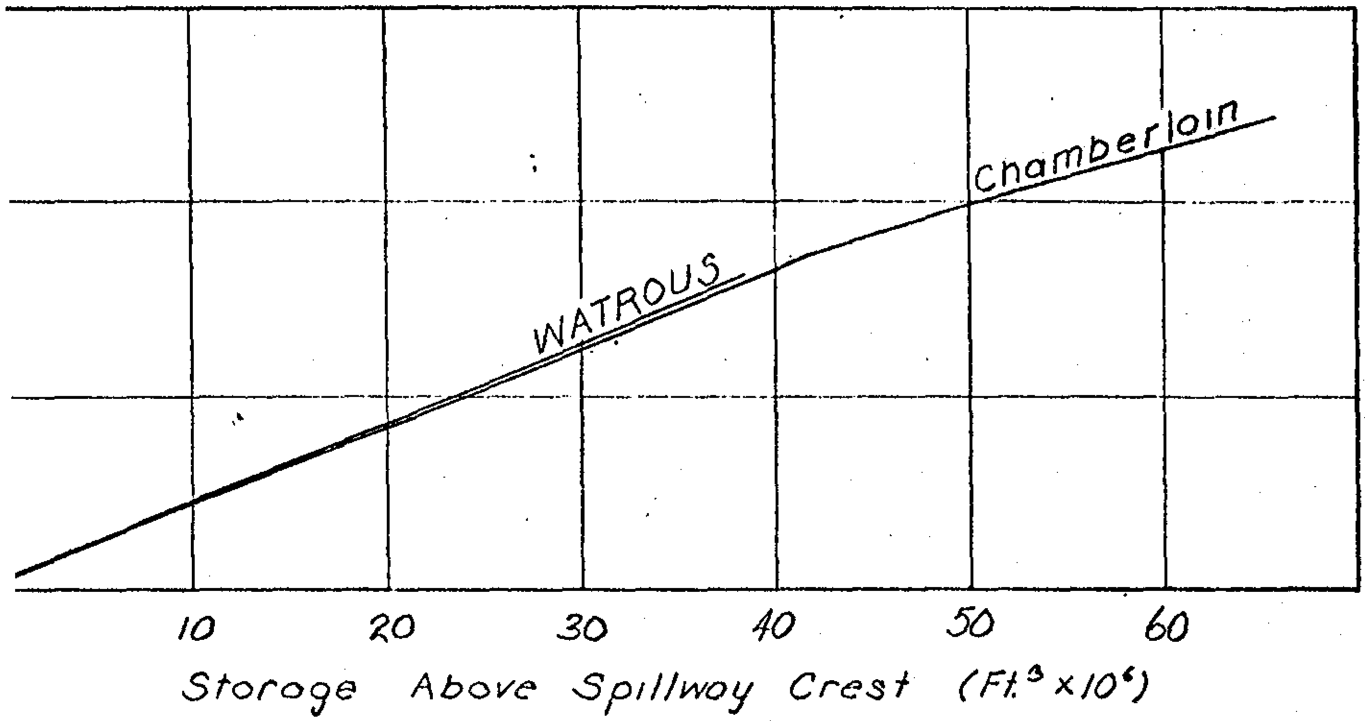
** Distributed and arranged as recommended in U. S. Department of the Interior Publication "Design of Small Dams"

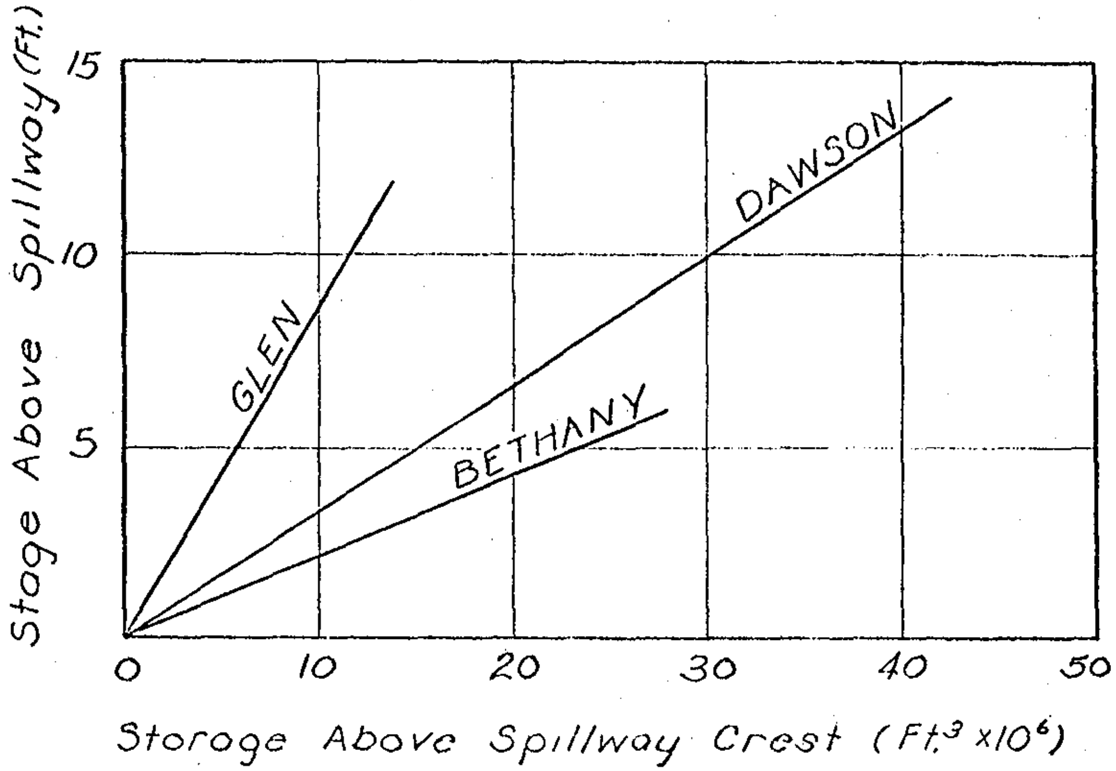
EXHIBIT 2





SPILLWAY RATING CURVES





STAGE - CAPACITY CURVES

MALCOLM PIRNIE ENGINEERS
220 WESTCHESTER AVENUE
WHITE PLAINS, N.Y. 10604

BY BM DATE 6/28/67

NEW HAVEN – MAX. POSSIBLE INFLOW HYDROGRAPH

JOB NO. 14470 SHEET NO. 1 OF 8

BETHANY – MAX. POSSIBLE STORM INFLOW HYDROGRAPH

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	
1" STORM	20	200	415	370	280	190	120	80	60	40	20	10	5	3	1	0						
0.3	6	60	125	111	84	57	36	24	18	12	6	3	2	1	0	0						
1.8		36	360	747	656	504	342	216	144	108	72	36	18	9	5	2	0					
2.5			50	500	1038	925	700	475	300	200	150	100	50	25	13	8	3	0				
12.0				240	2400	4980	4440	3360	2280	1440	960	720	480	240	120	60	36	12	0			
2.5					70	700	1450	1300	980	665	420	280	210	140	70	35	18	11	4	0		
2.1						42	420	870	777	589	400	252	168	126	84	42	21	11	6	2	0	
1.7							34	340	706	629	476	323	204	136	102	68	34	17	8	5	2	0
0.4								8	80	166	143	112	76	48	32	24	16	8	4	2	1	0
0.1									2	20	42	37	28	19	12	8	6	4	2	1	0	0
24.4	6	96	535	1598	4248	7208	7422	6593	5287	3829	2674	1863	1236	744	438	247	134	63	42	10	3	

Σ1" = 1814 1,814 × 24.4 = 44,262 CHECKS TOTAL = 44,276

✓ CHAMBERLAIN

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21		
1" STORM	15	30	370	475	440	330	220	160	120	95	70	55	38	22	19	10	5	3	1	0			
0.3	5	24	111	143	132	99	66	43	36	29	21	17	12	7	5	3	2	1	0	0			
1.8		27	144	665	855	791	595	396	288	216	171	126	99	69	40	32	18	9	5	2	0		
2.5			38	200	925	1190	1100	825	550	400	300	238	175	138	95	55	45	25	13	8	3	0	
12.0				180	960	4440	5700	5290	3960	2640	1920	1440	1140	840	660	456	264	216	120	60	36	12	
3.5					53	280	1295	1660	1540	1155	770	560	420	332	245	193	133	77	63	35	18	11	4
2.1						32	168	777	997	925	694	461	336	252	200	147	115	80	46	38	21	10	6
1.7							26	136	629	807	748	560	374	272	204	162	119	94	65	37	31	17	8
0.4								6	32	148	190	176	132	88	64	48	38	28	22	15	9	7	4
0.1									2	8	37	48	44	33	22	16	12	10	7	6	4	2	2 1
24.4	5	51	293	1188	2925	6532	8950	9136	8034	6328	4851	2626	2732	2031	1535	1112	746	540	341	201	122	59	

Σ1" = 2527 2527 × 24.4 = 61658 CHECKS TOT: 61640

MALCOLM PIRNIE ENGINEERS
222 WESTCHESTER AVENUE
WHITE PLAINS, N.Y. 10604

BY BM DATE 6/28/6[illegible]

JOB NO. 144700 SHEET NO. 2 OF 8

PROJECT NEW HAVEN - MAX. POSSIBLE INFLOW HYDROGRAPH

[illegible] MAX POSSIBLE STORM INFLOW HYDROGRAPH

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1"STM	20	130	269	220	140	80	50	30	20	10	5	3	2	1	0						
0.3	6	54	81	66	42	24	15	9	6	3	2	1	0	0	0						
1.8		36	324	486	396	252	144	90	54	36	18	12	6	0	0						
2.5			50	450	672	550	350	200	125	75	50	25	13	8	5	3	0				
12.0				240	2160	3230	2640	1680	960	600	360	240	120	60	36	24	12	0			
3.5				70	630	942	770	490	280	175	105	70	35	18	10	7	4	0	0		
2.1					42	378	569	461	294	168	105	63	42	21	11	6	4	2	0		
1.7						34	306	458	374	233	136	85	51	34	17	8	5	3	2	0	
0.4							8	72	107	88	56	32	20	12	8	4	2	1	1	0	0
0.1								2	18	27	22	14	8	5	3	2	1	0	0	0	0
24.4	6	96	455	1312	3942	5410	4802	3462	2218	1410	854	542	295	158	90	54	28	6	3		

$\Sigma 1" = 1030$ $1030 \times 24.4 = 25132$ CHECKS TOT. = 25137

WATROUS

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	
1"STM	20	220	440	420	320	220	150	110	80	60	40	25	15	10	5	1	0					
0.3	6	66	132	126	96	66	45	33	24	18	12	8	5	3	2	0	0					
1.8		36	396	792	756	576	396	270	198	144	105	72	48	30	18	12	0	0				
2.5			50	550	1100	1050	800	550	375	275	200	150	100	63	38	25	13	3	0			
12.0				240	2640	5280	5040	3840	2640	1800	1320	960	720	480	300	180	120	60	12			
3.5					70	770	1540	1470	1120	770	525	385	280	210	140	88	53	35	18	4		
2.1						42	462	924	881	671	462	315	231	168	126	84	53	32	21	10	2	
1.7							34	374	749	714	544	374	255	187	136	102	68	43	25	17	8	2
0.4								8	88	176	168	128	88	60	44	32	24	16	10	6	4	2
0.1									2	22	44	42	32	22	15	11	8	6	4	3	2	1
24.4	6	102	578	1708	4662	7784	8317	7469	6076	4596	3323	2434	1759	1223	819	534	286	195	90	40	16	5

$\Sigma 1" = 2136$ $2136 \times 24.4 = 52118$ CHECKS TOTAL = 52076

B-137

DAWSON - MAXIMUM POSSIBLE INFLOW HYDROGRAPH

TIME	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
1"STM	10	135	170	90	50	30	20	2	1	0											
0.3	3	41	51	27	15	9	6	1	0	0											
1.8		18	246	306	162	90	54	36	6	0											
2.5			25	338	425	225	125	75	50	5	3										
12.0				120	1620	2040	1080	600	360	240	40	12	0								
3.5					35	472	595	315	175	105	70	7	3								
2.1						21	284	257	189	105	63	42	4	2	0						
1.7							17	230	289	153	85	51	34	3	2	0					
0.4							4	54	68	36	20	12	8	1	0	0	0				
0.1							1	14	17	9	5	3	2	0	0	0	0	0			
24.4	3	59	322	791	2277	2857	2166	1682	1154	653	286	127	48	6	2						

24.4 × 508 = 12,400 CHECKS TOTAL = 12,440

NEW HAVEN - MAX. POSSIBLE INFLOW HYDROGRAPH

BY BM DATE 6/28/67

220 WESTCHESTER AVENUE, WHITE PLAINS, N.Y. 10604

JOB NO. 14700 SHEET NO. 3 OF 8

SUBJECT ROUTING MAX POSSIBLE STORM DISTRIBUTED ACCORDING TO "SMALL DAMS" THROUGH BETHANY

BETHANY RES. & SPILLWAY

TIME	I TOT.	$\frac{2S_1}{t} - O$	$\frac{2S_2}{t} + O_2$	O_2	STAGE
0	0	0	6	-	
1.0	6	4	106	1	-
2.0	96	86	717	10	-
3.0	535	667	2800	25	0.1
4.0	1598	2360	8206	220	0.7
5.0	4248	28080	37,328	1,210	2.5
5.25	4988	29628	40,344	1,350	2.8
5.50	5728	36244	48,440	2,050	3.6
5.75	6468	41340	55,016	3,550	4.3
6.00	7208	44216	58,685	5,400	4.8
6.25	7261	45485	60,060	6,600	5.1
6.50	7314	45660	60,341	7,200	5.2
6.75	7367	45741	60,530	7,300	5.2
7.00	7422	45830	60,466	7,350	5.2
7.25	7214	45766	59,987	7,350	5.2
7.50	7007	45787	59,594	7,100	5.2
7.75	6800	45694	59087	6,950	5.1
8.0	6593	6900	18,780	6,800	5.1
9.0	5287	7380	16,496	5,700	4.9
10.0	3829	7896	14,399	4,300	4.5
11.0	2674	8099	12,636	3,150	4.2
12.0	1863	7836	10,935	2,400	3.9
13.0	1236	6995	8,975	1,970	3.5
14.0	744			1,400	2.8

BM DATE 6/20/67

BY DATE

WESTCHESTER AVENUE
WHITE PLAINS, N. Y.

SHEET NO. 3 OF 8

JOB NO. 144700

...CT ROUTING MAX. POSSIBLE STORM DISTRIBUTED ...CORDING TO "SMALL DAMS" THROUGH CHAMBERLAIN RES. & SPILLWAY

CHAMBERLAIN RES. & SPILLWAY

TIME	I_{TOT}	$2S_1/t - O$	$2S_2/t + O$	O	STAGE ON SPILLWAY
0	0	0	5	-	-
1	5	3	59	1	-
2	51	39	383	10	-
3	293	343	1824	20	0.2
4	1,188	1524	5637	150	0.8
5	2925	4637	14394	500	2.0
6	6832	10,394	26,176	2000	4.8
7	8950	16,376	34,464	4900	8.4
8	9138	21,464	38,636	6500	10.1
9	8034	24,216	38,578	7210	10.8
10	6328	24,178	35,357	7200	10.8
11	4851	22,017	30,494	6670	10.2
12	3626	18,894	25,252	5800	9.3
13	2732	15852	20,615	4700	8.2
14	2031	13,815	17,381	3400	6.7
15	1535	12,181	14,828	2600	5.7
16	1112	10,728	12,586	2050	4.9
17	746	9,386	10,672	1600	4.1
18	540	8,232	9,113	1220	3.4
19	341	7,133	7,675	990	3.0
20	201	6095	6,418	790	2.7
21	122	4895	5,076	600	2.2'
22	59	4075	4158	410	1.7'
23	24	3458		350	1.5'

B.M. DATE 7/13/67

MALCOLM PIRNIE ENGINEERS
220 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

EXHIBIT 4

SHEET NO. 6 OF 8

D. BY .... DATE ....

JOB NO. 144700

JECT ROUTING MAX POSSIBLE STORM DISTRIBUTED ACCORDING TO "SMALL DAMS" THRO' GLEN RES. & SPILLWAY

GLEN RES. & SPILLWAY

TIME	I GLEN	I CHAMB	I TOTAL	$\frac{2S}{t} - O$	$\frac{2S}{t} + O$	O TOT	$O_{B.O.}$	$O_T - O_{B.O}$	STAGE ON SPILLWAY
0	0	0	0	0	7				
1	6	1	7	5	112	1	-	1	
2	90	10	100	72	647	20	-	20	
3	455	20	475	297	2234	175	-	175	0.8
4	1312	150	1462	1054	6958	590	-	590	2.2
5	3942	500	4442	1758	13610	2600	-	2600	6.5
6	5410	2000	7410	-590	16522	7100	285	6815	10.1
7	4802	4900	9702	-2478	17186	9500	"	9115	11.0
8	3462	6500	9962	-2514	16476	9950	"	9665	11.3
9	2218	7210	9428	-2324	15714	9500	"	9215	11.0
10	1410	7200	8610	-1746	14388	8730	"	8445	10.8
11	854	6670	7524	-972	12894	7680	"	7395	10.4
12	542	5800	6342	-106	11231	6500	"	6215	10.0
13	295	4700	4995	811	9364	5210	"	4925	9.5
14	158	3400	3558	1364	7612	4000	"	3715	8.3
15	90	2600	2690	1412	6206	3100	"	2815	6.9
16	54	2050	2104	1406	5138	2400	"	2015	5.5
17	28	1600	1628	1338	4192	1900	"	1615	4.8
18	6	1220	1226	1192	3411	1500	"	1215	3.9
19	3	990	993	1011	2794	1200	"	915	3.0
20	-	790	790	814	2204	990	"	705	2.5
21	-	600	600	604	1614	800	"	515	2.0
22	-	410	410	414	1174	600	"	315	
23	-	350	350			420	"	135	

B.M. DATE 7/14/67

223 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

SHEET NO. 7 OF 8

D. BY DATE

JOB NO. 14700

ECT ROUTING MAX POSSIBLE STORM ACCORDING TO "SMALL DAMS" THROUGH WATROUS

WATROUS RES. & SPILLWAY

TIME	I_{BETH}	$I_{WATROUS}$	I_{TOT}	$\frac{2S_1}{t} - O$	$\frac{2S_2}{t} + O$	O_2	STAGE
0	0	0	0	0	7		
1	6	1	7	5	124	1	-
2	102	10	112	44	759	40	-
3	578	25	603	559	3090	100	-
4	1708	220	1928	2290	10 090	400	1.0
5	4662	1210	5872	7090	25446	1500	3.1
6	5400	7784	13184	56,000	83701	8500	6.1
6.25	6600	7917	14,517	57,701	87468	13000	6.8
6.50	7200	8050	15,250	59,068	89801	14200	6.9
6.75	7300	8183	15483	59,600	90,150	15100	7.0
7.0	7350	8317	15,667	59,750	90,872	15200	7.0
7.25	7350	8105	15,455	60,072	90,520	15400	7.1
7.50	7100	7893	14,993	59,920	89,544	15300	7.1
7.75	6950	7681	14,631	59,514	88.444	15,000	7.0
8.0	6800	7469	14,269	3,300	29,345	14,700	7.0
9.0	5700	6076	11,776	4,345	25,611	12,200	6.7
0.0	4300	4590	8890	7,211	22,634	9,200	6.3
1.0	3150	3383	6533	9,234	20,601	6,700	6.1
2.0	2400	2434	4834	9,601	18,164	5,500	5.6
3.0	1970	1759	3729	9964	16,316	4,100	5.2
4.0	1400	1223	2623			3,200	4.8

B-142

MALCOLM [illegible] ENGINEERS
226 WESTCHESTER AVENUE
WHITE PLAINS, N. Y. 10604

B.M. DATE 7/14/67 — SHEET NO. 8 OF 8

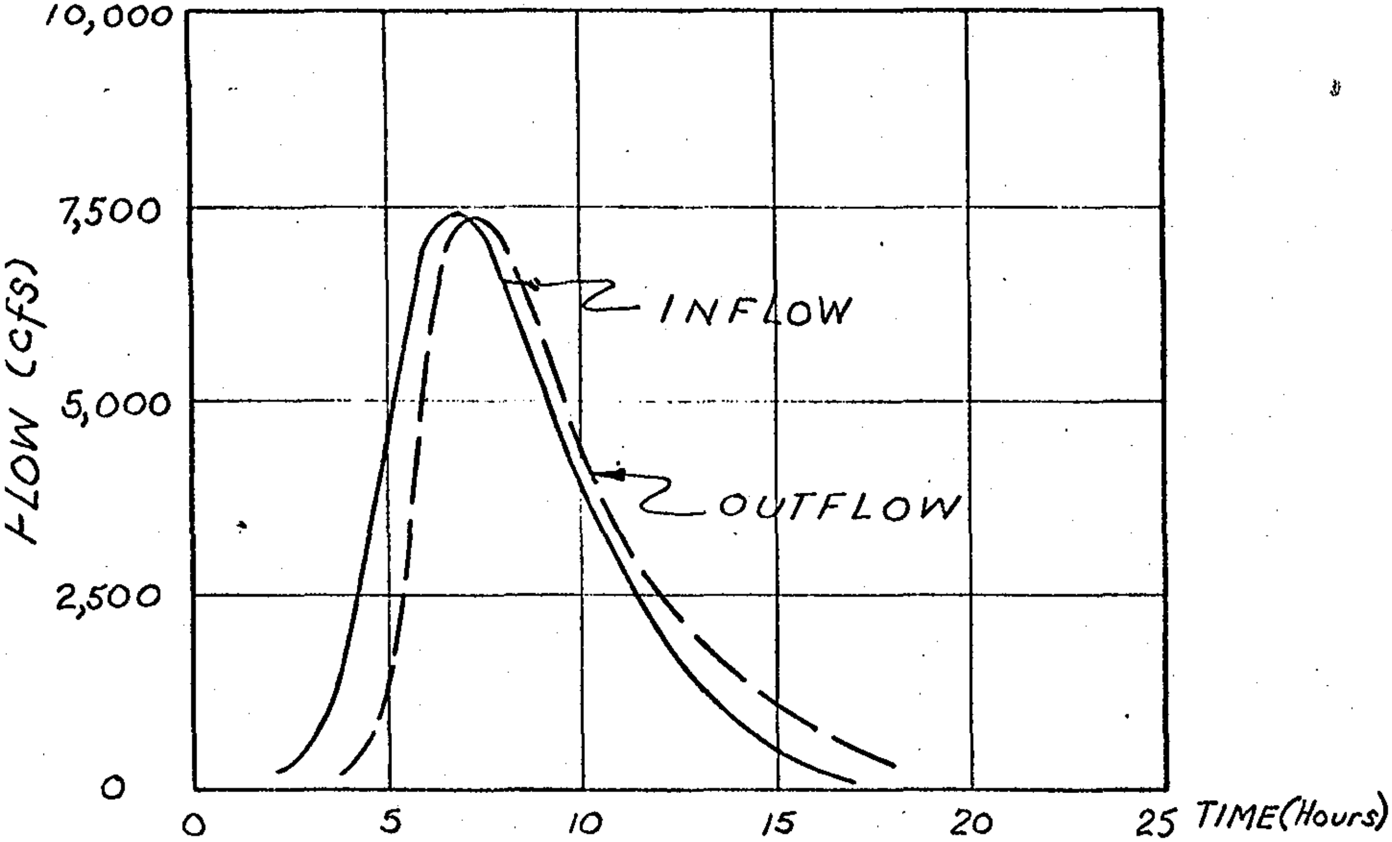
[illegible] BY .... DATE .... — JOB NO. 144700

[SUBJ]ECT: ROUTING MAX. POSSIBLE STORM, DISTRIBUTED ACCORDING TO "SMALL DAMS" THROUGH DAWSON

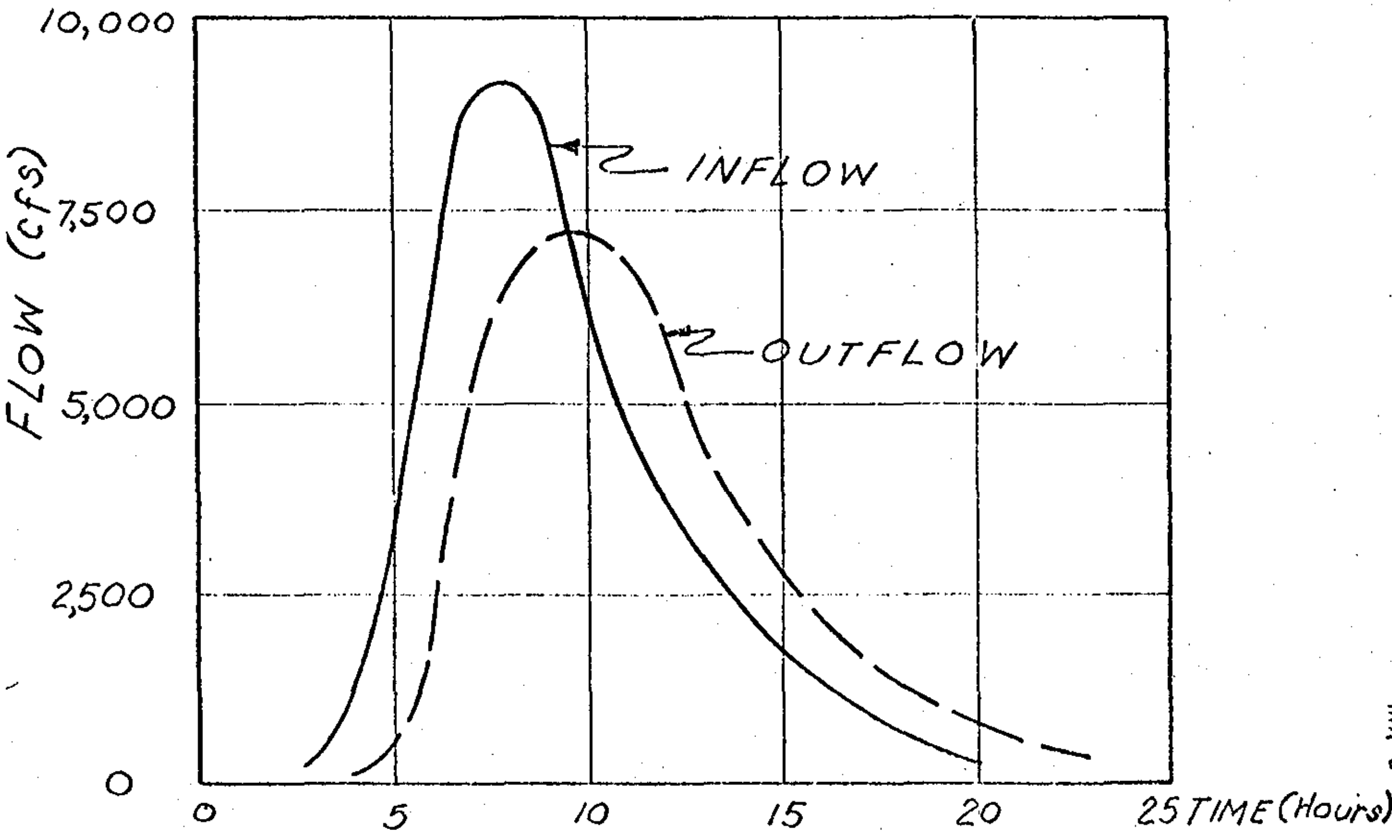
DAWSON RES & SPILLWAY

TIME	I_{TOT}	$\frac{2S_1}{t} - O_1$	$\frac{2S_2}{t} + O_2$	O_{TOT}	O_{BO}	$O_{TOT} - O_{BO}$	STAGE
0		0	1				
0.5	1	1	7	0	–	0	–
1.0	5	5	39	1	–	1	–
1.5	29	35	183	2	–	2	–
2.0	119	163	502	10	–	10	–
2.5	220	402	1219	50	–	50	0.6
3.0	597	919	2488	150	–	150	0.9
3.5	972	1888	4641	300	–	300	1.3
4.0	1781	3841	9256	400	–	400	1.4
4.5	3634	7256	17267	1000	–	1000	2.6
5.0	6377	12,667	30,457	2300	–	2,300	4.6
5.5	11,413	19,457	49,322	5,500	–	5,500	7.4
6.0	18,457	62,600	104,442	11,700	740	10,960	11.2
6.25	23,385	65,842	114,239	19,300	"	18,560	12.8
6.50	25,012	65,239	116,590	24,500	"	23,760	13.5
6.75	26,339	65,190	118,395	25,700	"	24,960	13.7
7.0	26,866	64,795	119,718	26,300	"	25,560	13.8
7.25	27,057	65,718	119,723	27,000	"	26,260	13.9
7.50	26,948	65,723	119,310	27,000	"	26,260	13.9
7.75	26,639	65,910	118,881	26,700	"	25,960	13.8
8.0	26,332	65,681	117,468	26,600	"	25,860	13.8
8.25	25,455	64,868	114,911	26,300	"	25,560	13.8
8.50	24,588	65,311	113,620	24,800	"	24,060	13.6
8.75	23,721	64,420	112,045	24,100	"	23,360	13.5
9.0	22,854	21,400	64,979	23,500	"	22,760	13.4
9.5	20,725	22,979	62,237	21,000	"	20,260	13.0
10.0	18,583	23,887	59,094	19,200	"	18,460	12.8
10.5	16,624	24,494	55,784	17,300	"	16,560	12.4
11.0	14,666	25,384	53,240	15,200	"	14,460	12.0
11.5	13,190	25,240	51,430	14,000	"	13,260	11.9
12.0	13,000			12,500	"	12,060	11.6
12.5							

B-143

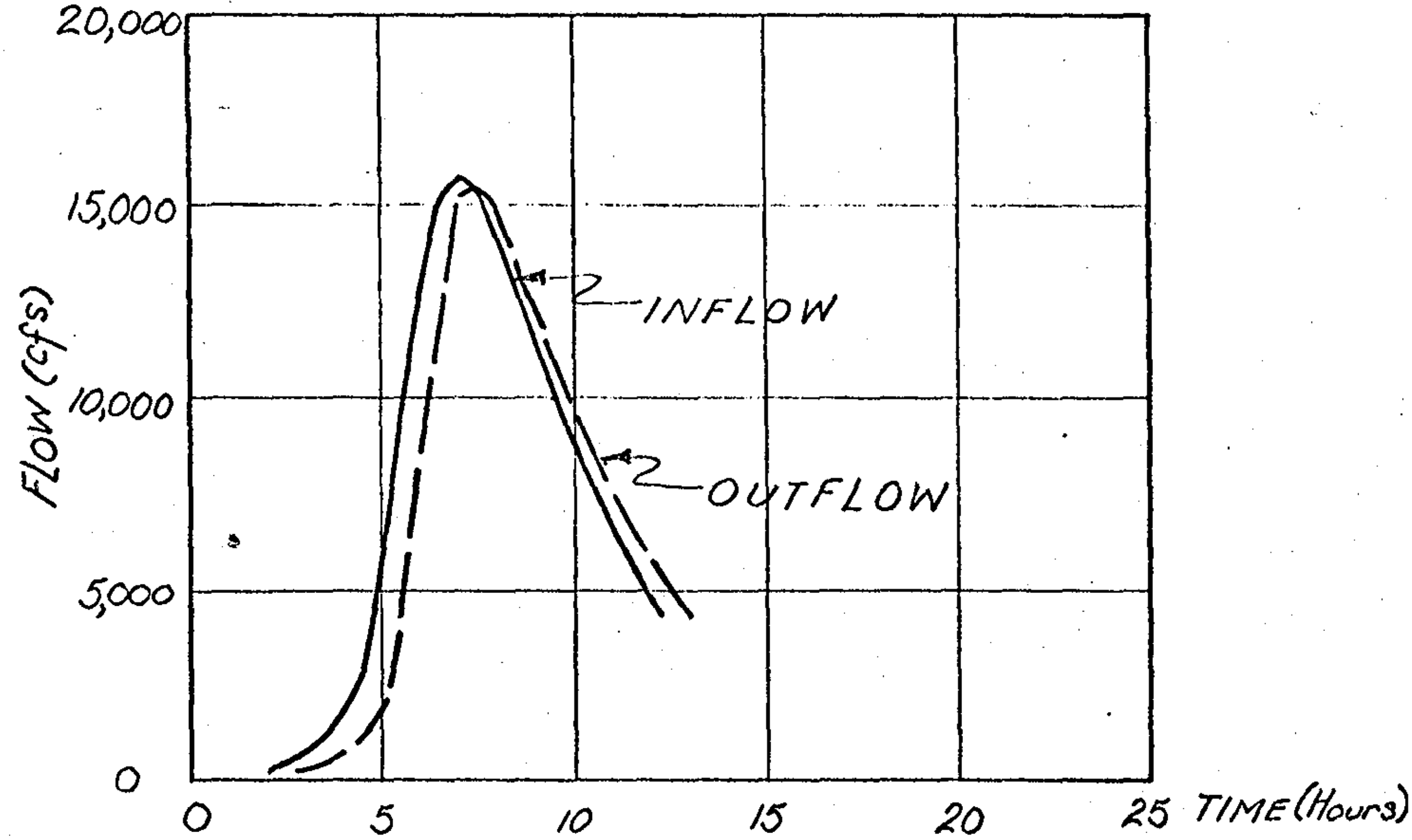


BETHANY

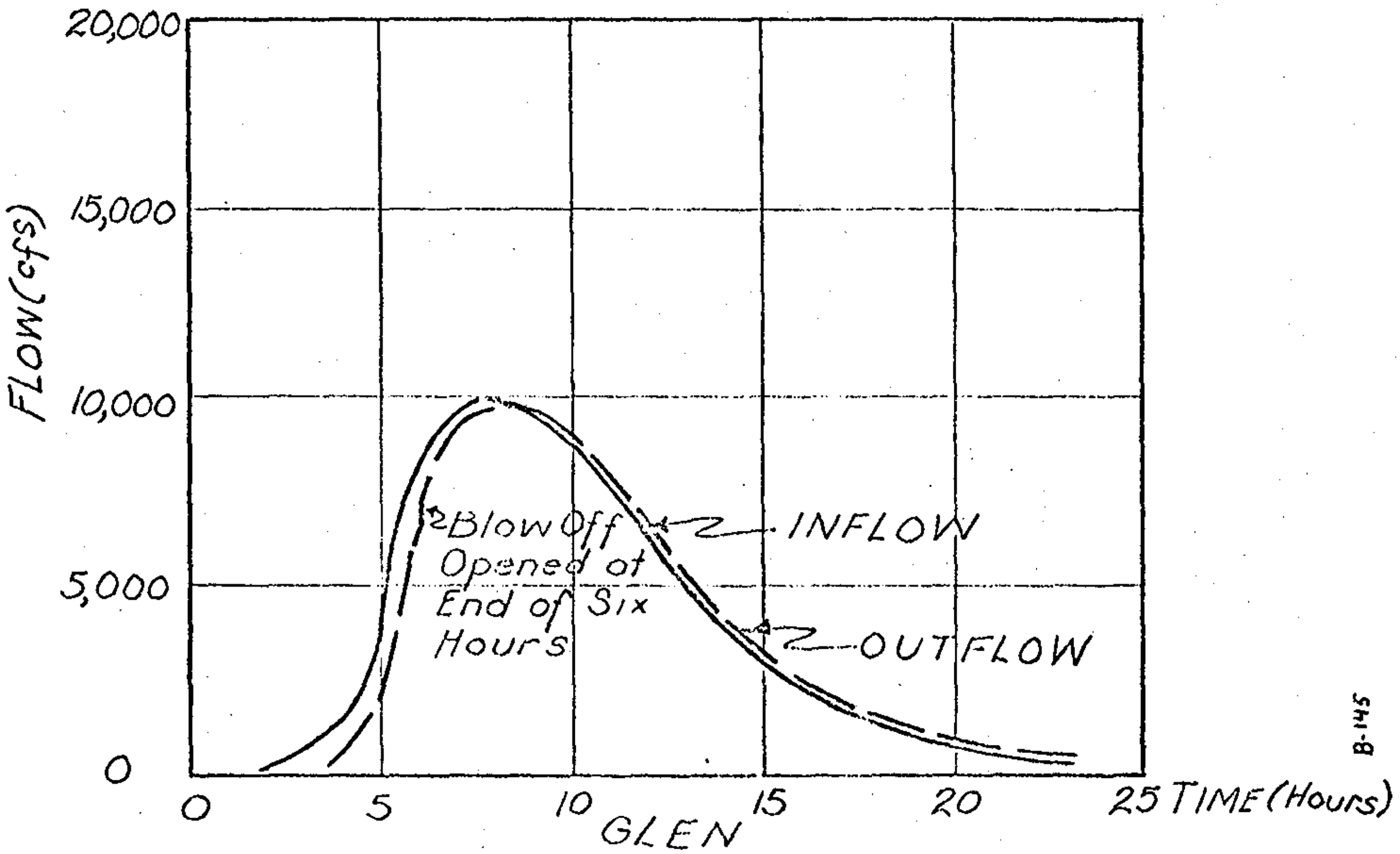


CHAMBERLAIN

B-144



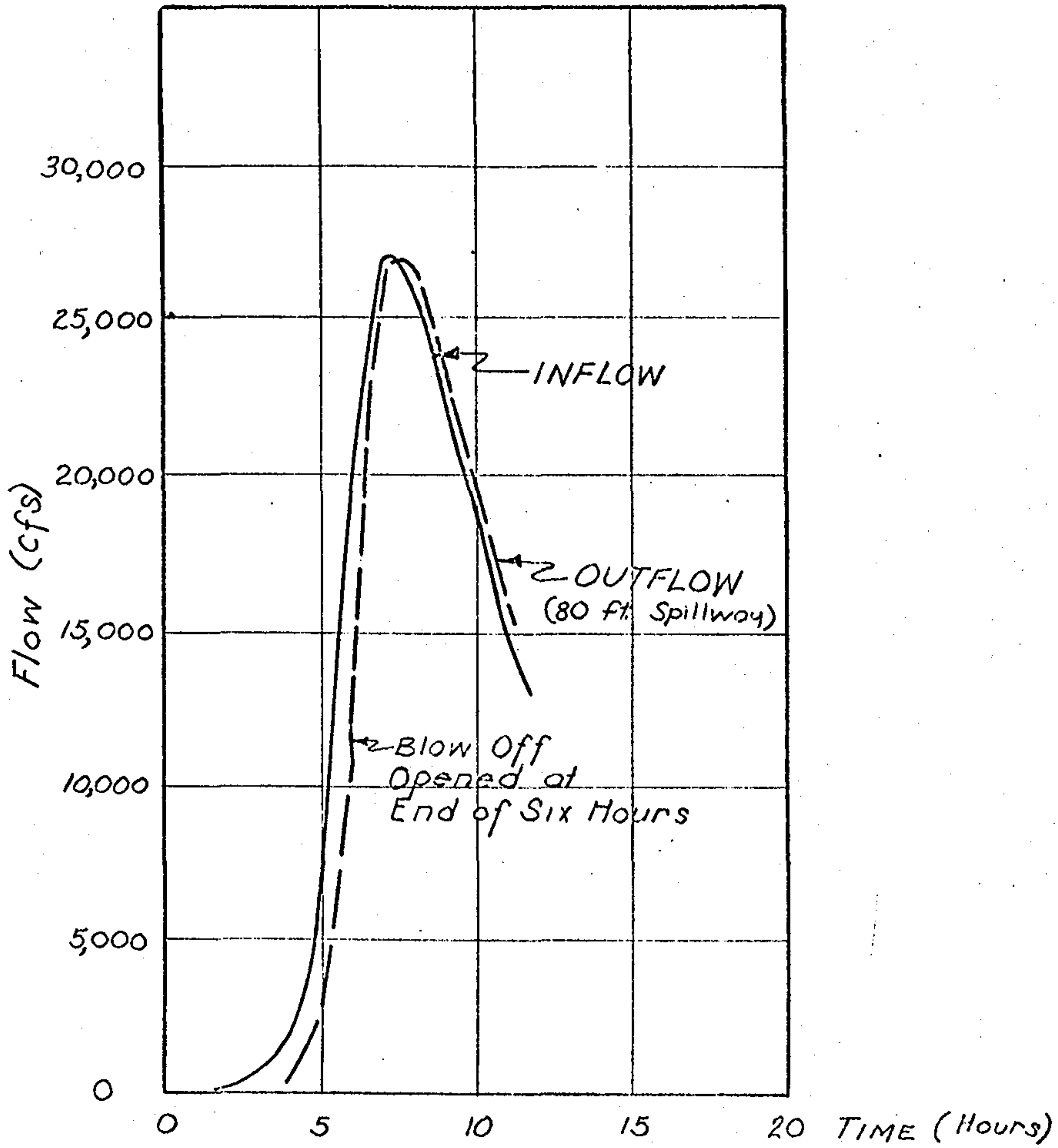
WATROUS



GLEN

B-145

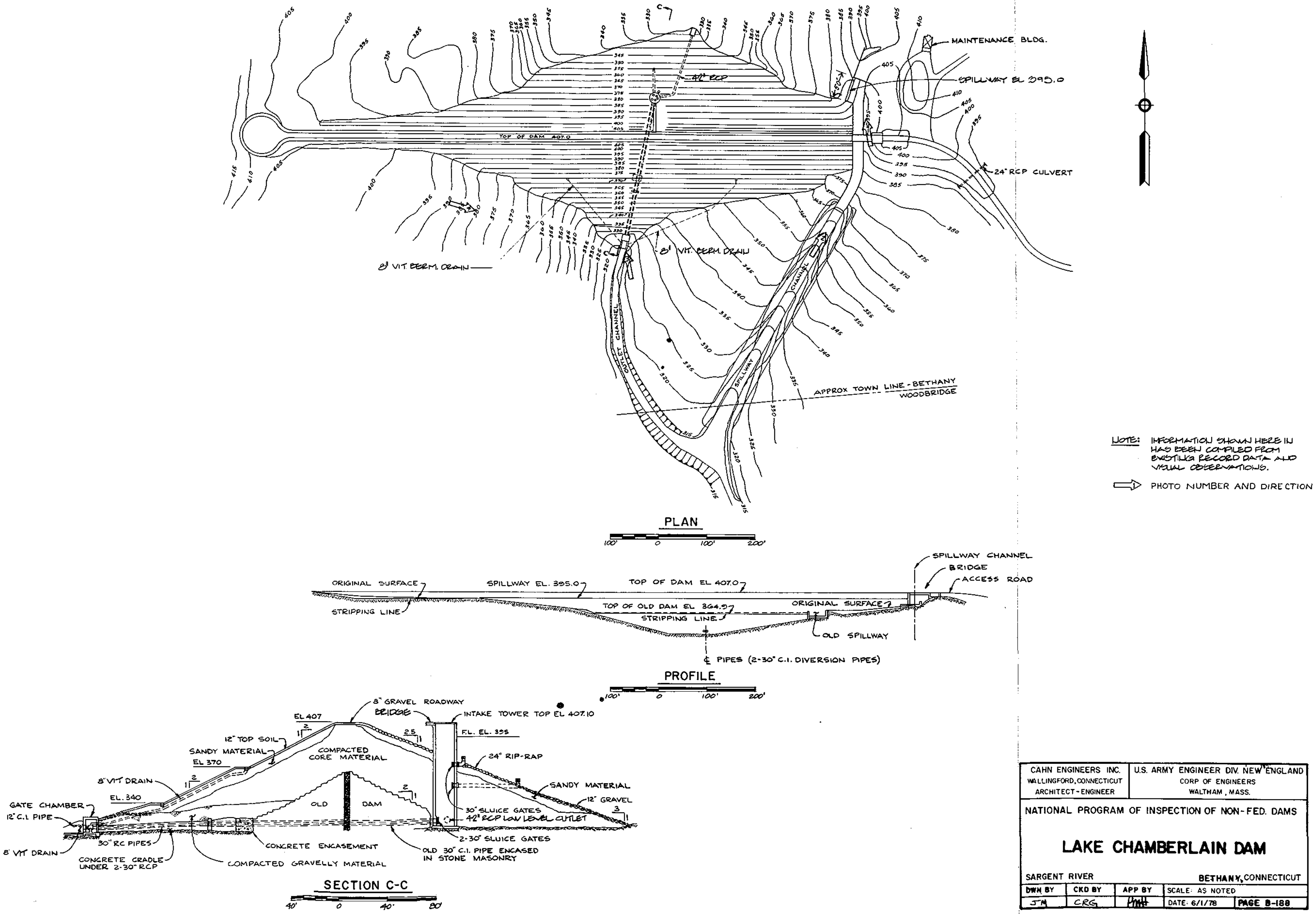
INFLOW - OUTFLOW



~~BETHANY~~
DAWSON

B-146

INFLOW - OUTFLOW



CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ARCHITECT-ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORP OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LAKE CHAMBERLAIN DAM

SARGENT RIVER — BETHANY, CONNECTICUT

DWN BY	CKD BY	APP BY	SCALE: AS NOTED
JM	CRG	[illegible]	DATE: 6/1/78 — PAGE B-188

APPENDIX

SECTION C: DETAIL PHOTOGRAPHS

PHOTO NO.1 - Spillway and channel walls. Note cracks and efflorescence between panel joints.

PHOTO NO.2 - Spillway channel cut into natural rock formation with spillway and bridge in background.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	LAKE CHAMBERLAIN DAM SARGENT RIVER BETHANY, CONNECTICUT
CAHN ENGINEERS INC. WALLINGFORD, CONN. ARCHITECT—— ENGINEER		CE # 27 531 GC DATE 6/1/78 PAGE C-1

PHOTO NO.3 - Outlet structure.

PHOTO NO.4 - Seep and crushed stone downstream at right end of dam.

APPENDIX

SECTION D: HYDRAULIC/HYDROLOGIC COMPUTATIONS

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

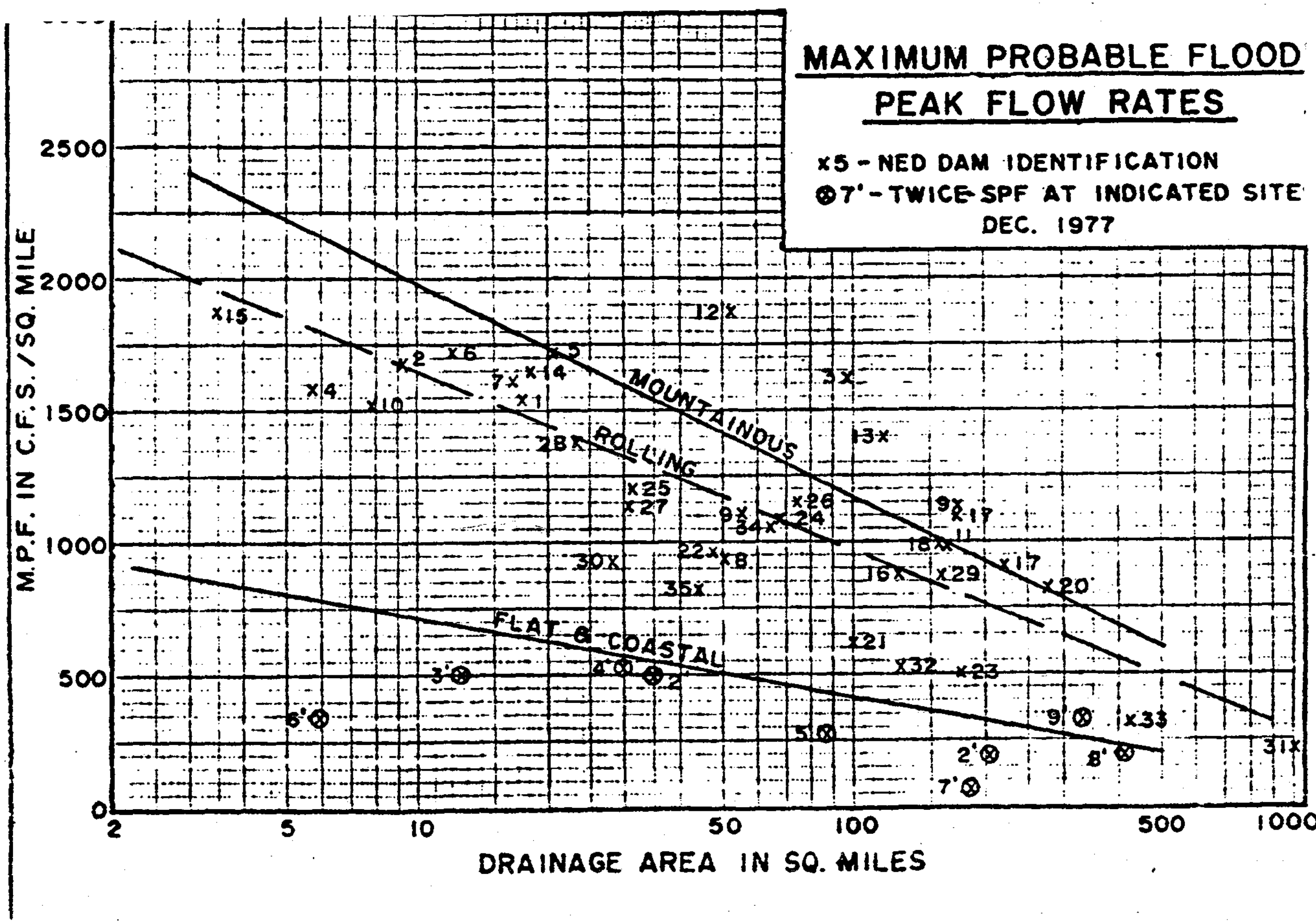
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

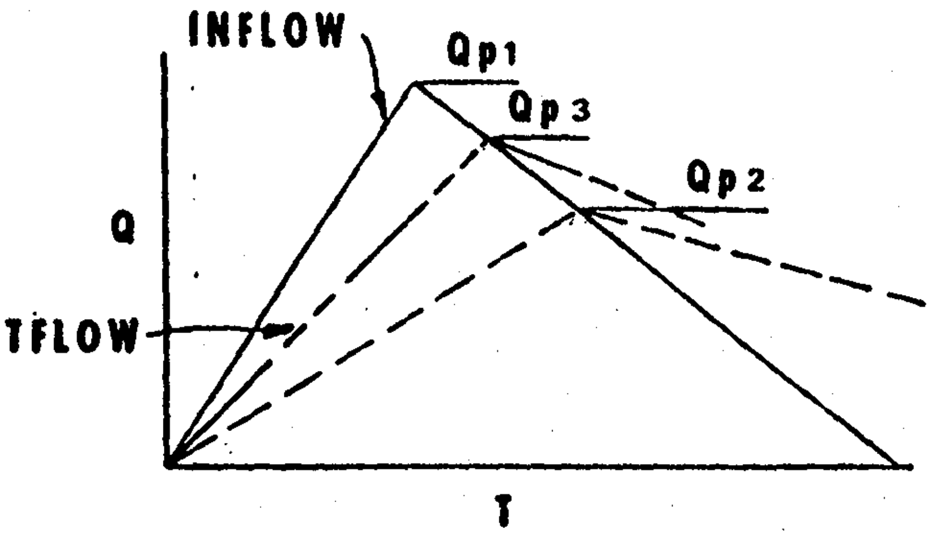
MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD (Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330



MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITE
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
MOUNTAINOUS
ROLLING
FLAT & COASTAL

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

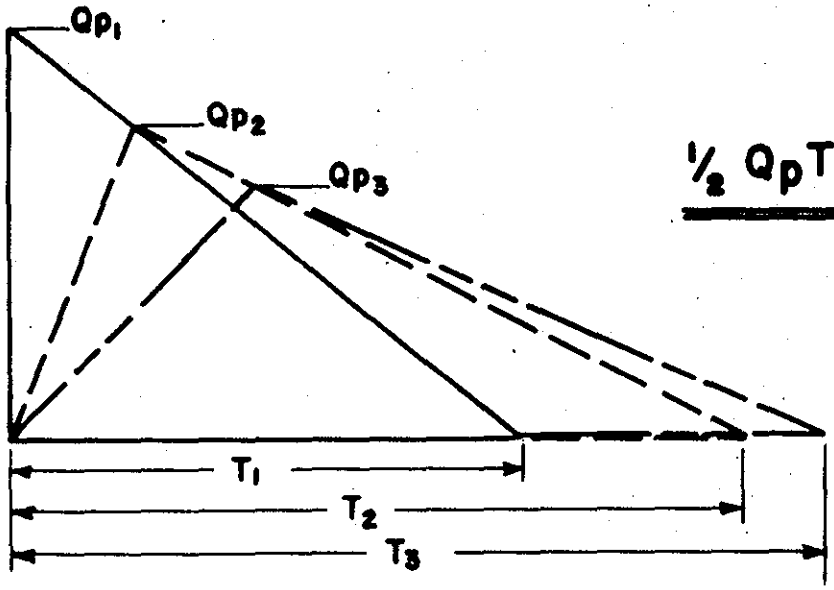
c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\,S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Q_{p1} = \frac{8}{27} W_b \sqrt{g}\, Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Q_{p2}(\text{TRIAL}) = Q_{p1}\left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Q_{p2} = Q_{p1}\left(1 - \frac{V_{AVG}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN, WOODBRIDGE, CONNECTICUT

(1) MAXIMUM PROBABLE FLOOD – PEAK FLOOD RATE

(a) WATERSHED CLASSIFIED AS "ROLLING" TYPE.

USE MPF GUIDE CURVES FURNISHED BY THE ACE NEW ENGLAND DIV. OFFICE FOR DETERMINATION OF MPF. "ROLLING" CURVE IS USED.

(b) WATERSHED AREA: D.A. = 3.9 SQ.MI (NEW HAVEN WATER CO. DESIG REPORT CHAMBERLAIN LAKE DAM, JULY, 1958)

C.E. MEASURED 4.0 SQ. MI

J.W. CONE, INSPECT. REPORT 6/26/65: DA = 4.1 m

USE DA = 4.0 SQ. MI

(c) FROM GUIDE CURVES:

M.P.F ≈ 1900 CFS / SQ. MI

(d) M.P.F - PEAK INFLOW

$$Q = 1900 \frac{CFS}{SQ.MI} \times 4.0 = 7{,}600 \ CFS$$

(2) SPILLWAY DESIGN FLOOD (SDF)

(a) CLASSIFICATION OF DAM ACCORDING TO A.C.E. RECOMMENDED GUIDELINES:

i) SIZE[1] (IMPOUNDMENT): STORAGE (MAX) = 4120 AC.FT (INTERM.)

HEIGHT (STRUCT) = 88 FT (INTERM.)

[1] FROM NEW HAVEN WATER CO. DATA (AUG. 1974) AND CHAMBERLAIN LAKE DESIGN REPORT & DWGS (1958). - (U.S. INVENTORY OF DAMS - MAX. STORAGE 5355 AC-FT)
STORAGE TO SPILLWAY ±* 894 MG ≅ 2740 AC-FT.
AREA @ FLOW LINE 115 AC. SPILLWAY TO TOP OF DAM, FREEBOARD 12'
ADD. STOR. TO TOP OF DAM 115 × 12 ≅ 1380 AC-FT. ∴ MAX. STORAGE IS ± 4120 AC. FT. * NOTE: DESIGN REPORT CURVE SHOWS ± 917 MG.

THEREFORE, THE DAM IS CLASSIFIED AS "INTERMEDIATE"

HYDRAULIC / HYDROLOGIC INSPECTION

LAKE CHAMBERLAIN, WOODBRIDGE, CT

(2)(a) (Cont'd) SDF - CLASSIFICATION OF DAM

(ii) HAZARD POTENTIAL:

THIS DAM IS LOCATED U/S OF GLEN AND DAWSON LAKES (EARTH DAMS) AND OF URBAN DEVELOPEMENTS IN WOODBRIDGE AREA.

THEREFORE, IT IS CLASSIFIED AS OF "HIGH" HAZARD POTENTIAL.

(iii) SDF

ACCORDING TO ACE RECOMMENDED GUIDELINES, FOR A DAM OF INTERMEDIATE SIZE AND HIGH HAZARD POTENTIAL

$$S.D.F = M.P.F = \underline{\underline{7,600 \text{ CFS}}}$$

(3) EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES:

(a) PEAK INFLOW (S.D.F = M.P.F)

$$Q_{P_1} = 7,600 \text{ CFS}.$$

HYDROLOGIC / HYDRAULIC INSPECTION
LAKE CHAMBERLAIN, WOODBRIDGE, CONNECTICUT

(3) (CONT'D) - EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES.

(b) SURCHARGE HEIGHT TO PASS Q_{P_1}

NOTE: SEE NEW HAVEN WATER CO. CHAMBERLAIN LAKE DAM DESIGN REPORT AND SPILLWAY RATING CURVES, DATED JULY, 1958.

$$c \approx 3.9 \qquad L = 50' \qquad cL = 195$$

$$Q \approx 195\, H^{3/2}$$

$$\therefore H = \left(\frac{Q}{195}\right)^{2/3}$$

$\therefore$ @ $Q_{P_1} = 7{,}600$ CFS $H_1 \approx 11.5'$

FREEBOARD OF SPILLWAY CREST TO TOP OF DAM IS 12'.

SPILLWAY CAPACITY AT $H = 12'$, $Q \approx 8100$ CFS

THE DAM IS NOT OVERTOPPED @ MPF $\approx 7{,}600$ CFS AND SURCHARGE HEIGHT IS 11.5' ABOVE SPILLWAY CREST.

(c) VOLUME OF SURCHARGE

ASSUME NORMAL POOL ELEVATION 0.5' ABOVE SPILLWAY CREST.

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN, WOODBRIDGE, CONN

(3) (CONT'D) - EFFECT OF SURCHARGE STORAGE ON MPDS.

(c) VOLUME OF SURCHARGE

STOR. @ FLOWLINE ≈ 2740 Ac-ft (see pg. 1)

STORAGE ABOVE SPILLWAY 115 × 0.5 ≈ 60 Ac-ft

TOTAL: 2800 Ac-ft.

AREA OF POOL AT FLOWLINE = 115 Ac

∴ VOLUME OF SURCHARGE

$115 \times (11.5 - 0.5) \approx 1270$ Ac-ft

D.A. = 4.0 SQ. MI

$$S_1 = \frac{1270}{4.0 \times 53.3} = 5.96'' \text{ SAY } 6.0''$$

(d) PEAK OUTFLOW FOR SURCHARGE S_1

(SEE GUIDELINES FOR ASSUMING TRIANGULAR HYDROGRAPH: MPF RUNOFF IN NEW ENGLAND IS ±19")

$$Q_{p2} = Q_{p1}\left(1 - \frac{S_1}{19}\right)$$

$$Q_{p2} = 7{,}600\left(1 - \frac{6.0}{19}\right)$$

$$\approx 5200 \text{ CFS}$$

HYDROLOGIC/HYDRAULIC INSPECTION

LAKE CHAMBERLAIN, WOODBRIDGE, CONN

(3) (cont'd) - EFFECT OF SURCHARGE STORAGE ON MPDs.

(d) PEAK OUTFLOW FOR SURCHARGE S_1

@ $Qp_2 = 5,200$ CFS

$H_2 \approx 8.9'$

$S_2 \approx 4.5''$

$S_{AVE} \approx 5.2''$

(e) RESULTING PEAK OUTFLOW

$Qp_3 = 7,600\left(1 - \frac{5.2}{19}\right)$

$\approx 5,500$ CFS

@ $Qp_3 \approx 5,500$ CFS

$H_3 \approx 9.3'$

(f) SUMMARY:

PEAK INFLOW: $Qp_1 = MPF = 7,600$ CFS

PEAK OUTFLOW: $Qp_3 = 5,500$ CFS

AVERAGE SURCHARGE HEIGHT = 9.3 ft. ABOVE SPILLWAY CREST. TO ELEV ±* 407.6' m.s.L

NOTE: TOP OF DAM IS AT ELEV *410.3' MSL DAM WILL NOT BE OVERTOPPED FOR THIS PEAK INFLOW. (FREEBOARD ≈ 2.7')

* NEW HAVEN WATER CO. DATA GIVE ELEVATIONS IN NEW HAVEN DATUM (MEAN HIGH WATER)
MSL (USC&GS DATUM) = NEW HAVEN DATUM (MHW) + 3.31'

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE, CONN

DOWNSTREAM DAM FAILURE HAZARD

(1) ESTIMATE OF DOWNSTREAM DAM FAILURE HYDROGRAPH (SEE ACE "RULE OF THUMB" GUIDELINE FOR ESTIMATING THE HYDROGRAPHS)

(a) ESTIMATE OF RESERVOIR STORAGE AT TIME OF FAILURE (SEE D. SHEN COMPS. 5/17/1978)

(i) MAXIMUM STORAGE CAPACITY = 4120 Ac-ft.

AREA AT FLOWLINE = 115 Ac.

(ii) HEIGHT OF EMBANKMENT (ELEV. *410.3 MSL) SPILLWAY (ELEV. *398.3 MSL) = 12 ft.

(iii) HEIGHT OF MAXIMUM POOL FROM STREAM BED (ELEV. 322.3 MSL) = 88 ft.

(iv) ESTIMATE VOLUME OF STORAGE AT TIME OF FAILURE. TO SURCHARGE ELEV ± 407.6 MSL i.e. 9.3' ABOVE THE SPILLWAY CREST.

$$S \approx 2740 + 115\,(9.3)$$
$$\approx \underline{3{,}800 \text{ Ac-ft}} \qquad \frac{S}{2} = 1{,}900 \text{ ft}$$

*NOTE. NEW HAVEN WATER CO DATA GIVE ELEVATIONS IN NEW HAVEN DATUM (NHW)
MSL (USCGS DATUM) ≡ NEW HAVEN DATUM (NHW) + 3.31'

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE, CONN.

DOWNSTREAM DAM FAILURE HAZARD

(1) (Cont'd) ESTIMATE OF DOWNSTREAM DAM FAILURE HYDROGRAPH

(b) PEAK FAILURE OUTFLOW (Q_{p1})

(i) BREACH WIDTH:

FROM THE NEW HAVEN WATER CO. LAKE CHAMBERLAIN "AS-BUILT" PLANS, JULY, 1958 #10040

TOTAL LENGTH ALONG MID-HEIGHT ≈ 480 FT

$W \approx 0.4 \times (480) \approx 190'$

TAKE $W_b = \underline{\underline{190'}}$

(ii) TOTAL HEIGHT AT TIME OF FAILURE

$y_0 \approx 407.6 - 322.3 = \underline{\underline{85.3'}}$

APPROX. WAVE HEIGHT IMMEDIATE D/S OF DAM SITE

$y \approx 0.44\, y_0 \approx 38'$

(iii) PEAK FAILURE OUTFLOW Q_{p1}

$Q_{p1} = \frac{8}{27} \sqrt{g}\, W_b\, y_0^{1.5} \approx \underline{\underline{251,000 \text{ CFS}}}$

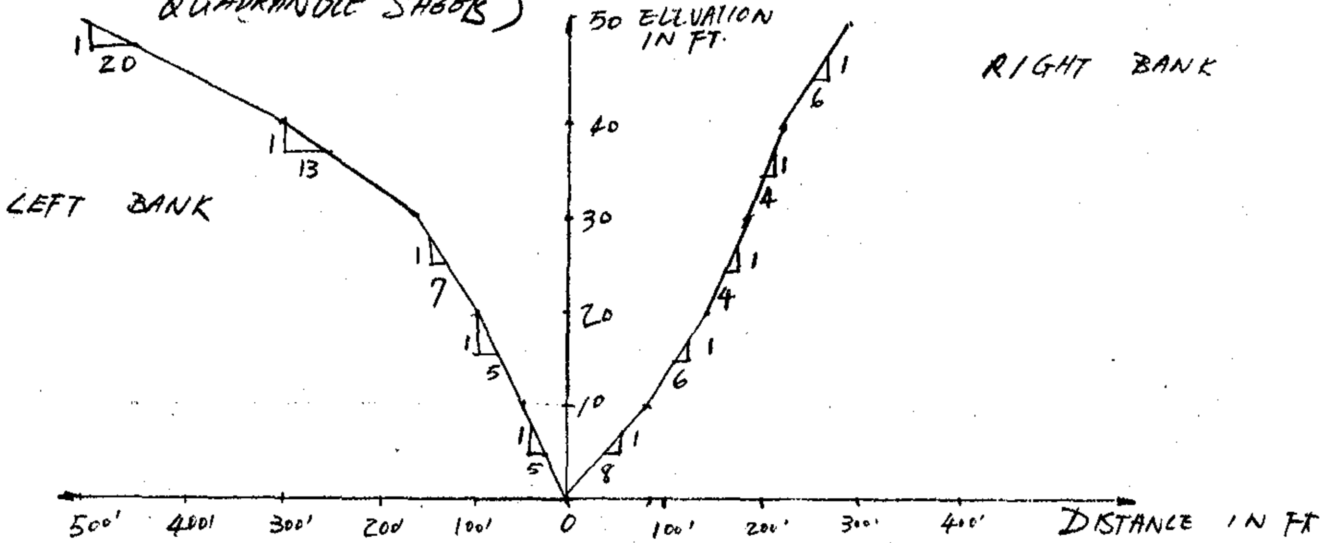
HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE, CONN.

DOWNSTREAM DAM FAILURE HAZARD

(1) (CONT'D) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(2) TYPICAL D/S CROSS-SECTION & RATING CURVES.
(FROM U.S.G.S. WOODBRIDGE AND NEW HAVEN QUADRANGLE SHEETS)



Assume (1) MANNING'S ROUGHNESS COEFFICIENT
$n \approx 0.050$

(2) AVG. SLOPE
$S \approx 0.018$ ft/ft.
$\sqrt{S} \approx 0.134$

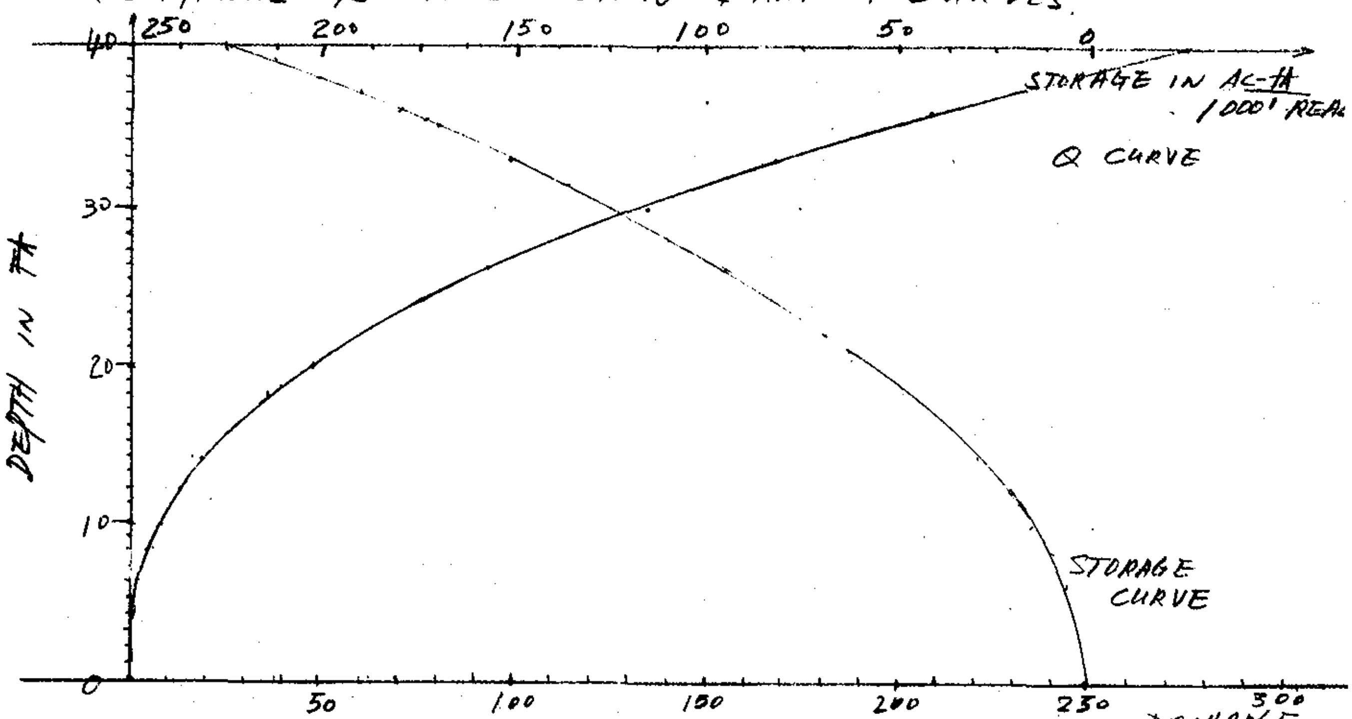
HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE, CONN

DOWNSTREAM DAM FAILURE HAZARD

(1) (cont'd) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(c) TYPICAL D/S CROSS SECTION & RATING CURVES.



(d) REACH OUTFLOW (Q_{P2})

(i) @ Q_{P1} = 251,000 CFS, FROM RATING CURVE

STAGE ≅ 38.8'

REACH DISTANCE FROM LK. CHAMBERLAIN OUTFALL TO U/S END OF GLEN LAKE IS APPROX 5600'

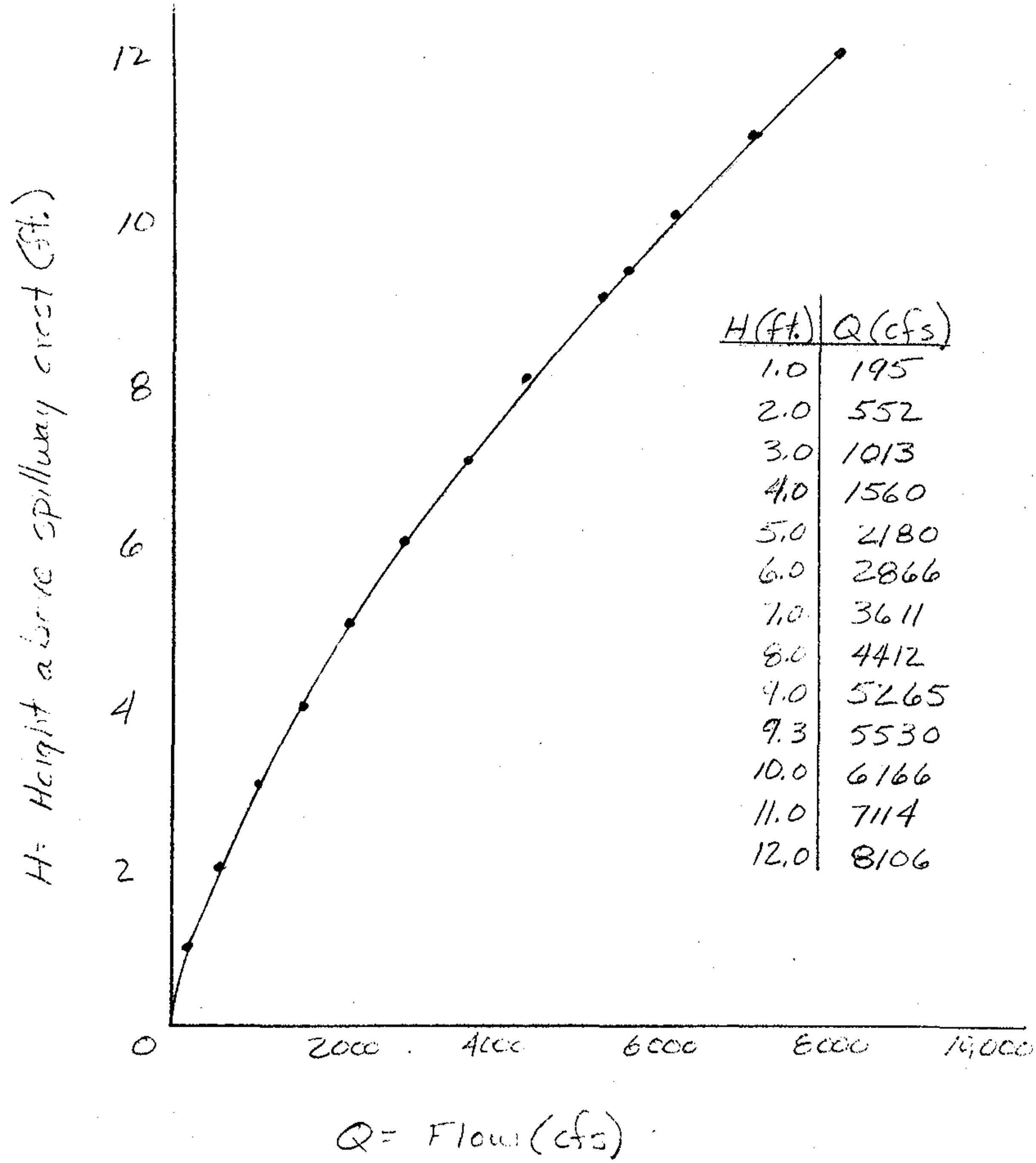
VOLUME IN REACH: $V_1 \cong 210 \times 5.6 \cong 1180$ AC-FT $< \frac{S}{2}$ O.K.

ahn Engineers Inc. Consulting Engineers

Project: Lake Chamberlain Dam | Sheet 1 of 1
Computed By: HM/CRG | Checked By: | Date: 9/6/78
Field Book Ref. | Other Refs. | Revisions

SPILLWAY RATING CURVE

$$Q = 195H^{3/2} \quad \text{FOR } H \leq 12'$$



H (ft.)	Q (cfs)
1.0	195
2.0	552
3.0	1013
4.0	1560
5.0	2180
6.0	2866
7.0	3611
8.0	4412
9.0	5265
9.3	5530
10.0	6166
11.0	7114
12.0	8106

ıhn Engineers Inc. **Consulting Engineers**

INSPECTION OF NON-FEDERAL DAMS IN NEW ENGLAND Sheet 5 of 8

ıd By D. SHEN Checked By [illegible] Date 6/1/1978

ook Ref. Other Refs. CE# 27-531-GC Revisions

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE, CONN

DOWNSTREAM DAM FAILURE HAZARD

(1) (cont'd) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(d) REACH OUTFLOW Q_{p2}

(ii) $Q_{p2} = Q_{p1}\left(1-\frac{V_1}{S}\right) = 251{,}000\left(1-\frac{1180}{3800}\right)$

$\approx 173{,}000$ CFS

(iii) @ $Q_{p2} \approx 173{,}000$ CFS

STAGE $\approx 33.4'$

$V_2 \approx 155 \times 5.6 \approx 870$ Ac-ft

(iv) AVE. VOLUME IN REACH $V_{AVE} = 1025$ Ac-ft

$Q_{p2} \approx 251{,}000\left(1-\frac{1025}{3800}\right)$

$\approx 183{,}000$ CFS

STAGE $\approx 34'$

Q_{p2}, & STAGE ARE FOR THE IMMEDIATE D/S REGION OF GLEN LAKE

(e) ESTIMATE EFFECT OF GLEN LAKE RESERVOIR ON Q_{p2}

(i) MAXIMUM SPILLWAY DISCHARGE (GLEN LAKE)

(SEE J.W. CONE 1965 REPORT CONCERNING DAMS OWNED BY THE NEW HAVEN WATER CO, ON THE WEST AND SARGENT RIVER)

LENGTH OF SPILLWAY = 40'

MAXIMUM FREEBOARD = 4'

ROUNDED-CRESTED OGEE TYPE

$C \approx 3.5$

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE, CONN.

DOWNSTREAM DAM FAILURE HAZARD

(1) (cont'd) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(2) ESTIMATE EFFECT OF GLEN LAKE ON Q_{p2}

(i) MAXIMUM SPILLWAY DISCHARGE ($H=4'$)

$$Q = 140 H^{3/2} \simeq 1{,}120 \text{ CFS}$$ (SEE J.W. CINE)

(ii) SURCHARGE HEIGHT ABOVE SPILLWAY CREST

ASSUME SPILLWAY DISCHARGE = INFLOW FLOOD $= Q_{p2} = 183{,}000$ CFS

TOTAL LENGTH OF THE GLEN LAKE DAM (330') AND SIDE SPILLS $\simeq 550'$

ASSUME $C \simeq 2.7$

$$Q \simeq (2.7)(550)(H-4)^{3/2}$$

$$\simeq 1485 (H-4)^{3/2}$$

THEREFORE,

$$Q \simeq 140 H^{3/2} + 1485 (H-4)^{3/2}$$

$\therefore$ @ $Q_{p2} \simeq 183{,}000$ CFS

$H_2 \simeq 27.0'$

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN WOODBRIDGE CONN

DOWNSTREAM DAM FAILURE HAZARD

(I) (CONT'D) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(2) ESTIMATE EFFECT OF GLEN LAKE ON Qp_2

(iii) EFFECT OF STORAGE OF GLEN LAKE

AREA OF LAKE AT FLOWLINE = 26 Ac (J. W. CONE REPORT, 1965)

∴ VOLUME OF SURCHARGE:

ASSUME NORMAL POOL DEPTH 0.5' ABOVE FLOWLINE

$V_R \simeq 26 \times (27.0 - 0.5) \simeq 690$ Ac-ft

(iv) PEAK FLOOD OUTFLOW, TRIAL Qp_3

$$Qp_3 = Qp_2 \left(1 - \frac{V_R}{S}\right) = 183{,}000 \left(1 - \frac{690}{3800}\right)$$

$$\simeq 150{,}000 \text{ CFS}$$

@ $Qp_3 \simeq 150{,}000$ CFS

$H_3 \simeq 24'$

$V_R \simeq 610$ Ac-ft

AVE. STORAGE $V_{AVE} = 650$ Ac-ft

HYDROLOGIC / HYDRAULIC INSPECTION

LAKE CHAMBERLAIN, WOODBRIDGE CT

DOWNSTREAM DAM FAILURE HAZARD

(b) (cont'd) ESTIMATE OF D/S DAM FAILURE HYDROGRAPHS

(e) EFFECT OF GLEN LAKE ON Qp_2

(v) PEAK OUTFLOW Qp_3

@ $V_{AVE} = 650$ AC-FT.

$$Qp_3 = Qp_2\left(1 - \frac{V_{AVE}}{S}\right) = 183{,}000\left(1 - \frac{650}{3800}\right)$$

$$\approx \underline{152{,}000 \text{ CFS}}$$

$H_3 \approx 24.4'$ SAY $\pm 24'$ ABOVE SPILLWAY CREST OR $\pm 20'$ ABOVE EMBANKMENT.

IT IS PROBABLE THAT GLEN LAKE DAM WILL FAIL UNDER THIS SURCHARGE CARRYING THE COMBINED FLOOD WAVE D/S TO LAKE DAWSON

(f) SUMMARY:

PEAK FAILURE OUTFLOW $Qp_1 = 251{,}000$ CFS

U/D OF GLEN LAKE $Qp_2 = 183{,}000$ CFS

STAGE $\approx 34'$

PEAK OUTFLOW FROM GLEN LAKE

$Qp_3 = 152{,}000$ CFS

SURCHARGE ABOVE SPILLWAY $H_3 \approx 24'$

GLEN LAKE DAM WILL BE OVERTOPPED BY $\pm 20'$

NOTE:

THESE COMPUTATIONS HAVE BEEN PERFORMED BASED UPON A DAM BREACH WITH A SURCHARGED WATER SURFACE ELEVATION. IN ACCORDANCE WITH NORMAL CORPS PROCEDURES, COMPUTATIONS ARE PERFORMED BASED UPON A WATER SURFACE ELEVATION AT THE TOP OF THE DAM. A DAM BREACH WITH THE WATER SURFACE AT THE TOP OF THE DAM AND WITHOUT HEAVY DOWN-STREAM CHANNEL FLOW COULD BE MORE CRITICAL THAN A DAM BREACH WITH A SURCHARGE. THE DIFFERENCE, IN THIS CASE, IS NOT SUBSTANTIAL.

APPENDIX

SECTION E: INVENTORY OF DAMS
IN THE UNITED STATES

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	306	NED	CT	009	05				LAKE CHAMBERLAIN DAM	4123.6	7259.3	08SEP78

POPULAR NAME	NAME OF IMPOUNDMENT
	LAKE CHAMBERLAIN

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY—TOWN—VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	07	SARGENT RIVER	WOODBRIDGE	2	4000

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
REERPG	1958	S	88	88	4120	2740	NED	N	N	N	N	22AUG78

REMARKS

D/S HAS	SPILLWAY CREST LENGTH	TYPE	WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	PROPOSED (MW)	NAVIGATION LOCKS NO.	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	1300	U	50	8100												

OWNER	ENGINEERING BY	CONSTRUCTION BY
NEW HAVEN WATER CO	MALCOLM PIRNIE ENG	C W BLAKESLEE + SONS INC

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
CAHN ENGINEERS INC	01JUN78	PL 92-367

REMARKS